SEWAGE AND INDUSTRIAL WASTES

(Formerly Sewage Works Journal)

JOURNAL of the FEDERATION OF SEWAGE AND INDUSTRIAL WASTES ASSOCIATIONS

Special Features

Improving Interceptor Efficiency—Riis-Carstensen

Oil Plant Waste Water Use—Sessler

Biotic Data Reporting—Beck

Pea-Blancher Waste Digestion—Oliver and Dunstan

Plankton Classification—Palmer and Ingram

Field Disposal of Beet Wastes—Porges and Hopkins

OCTOBER, 1955—VOL. 27, No. 10

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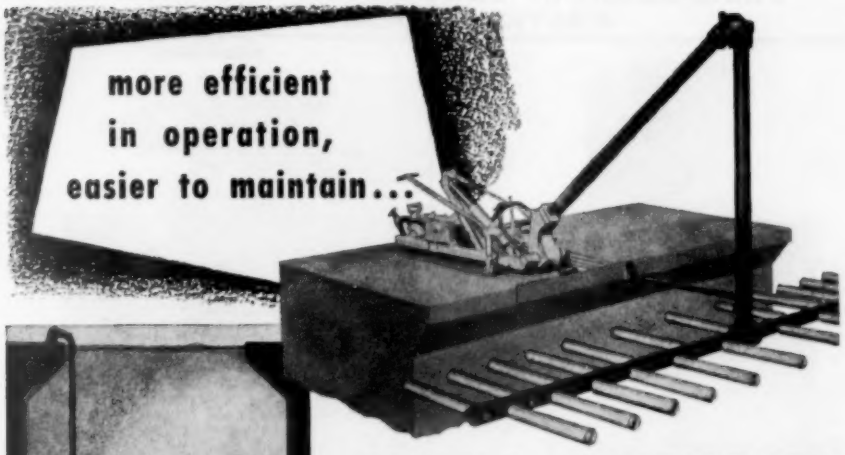
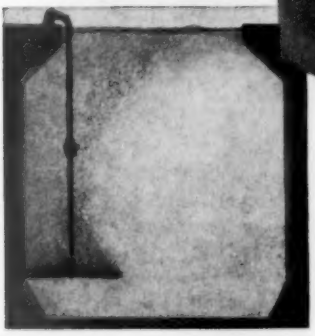
Please remit directly to your Member Association Secretary immediately upon receipt of the first dues notice from him.





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A monthly Journal devoted to the advancement of fundamental and practical knowledge concerning the nature, collection, treatment and disposal of sewage and industrial wastes, and the design, construction, operation and management of such works.

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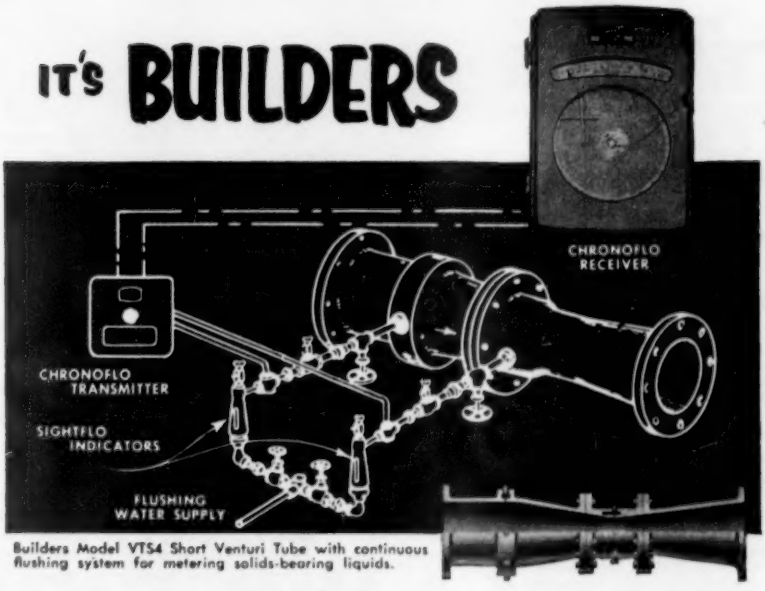

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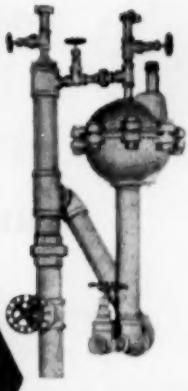

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cuts pumping costs

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- Reliable! Bronze seat with silicon rubber-tipped valve stem designed to prevent sticking shut or leaking water.
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Association	*Place*	*Time*
West Virginia Sewage and Industrial Wastes Assn.	Waldo Hotel, Clarksburg, W. Va.	Oct. 19–20, 1955
Pacific Northwest Sewage and Industrial Wastes Assn.	Empress Hotel, Victoria, B.C.	Oct. 27–29, 1955
Nebraska Sewage and Industrial Wastes Assn.	Yancey Hotel Grand Island, Nebr.	Nov. 3–4, 1955
Florida Sewage and Industrial Wastes Assn.	Orange Court Hotel Orlando, Fla.	Nov. 6–9, 1955
North Carolina Sewage and Industrial Wastes Assn.	Robert E. Lee Hotel Winston Salem, N. C.	Nov. 14–16, 1955

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in conjunction with

California Sewage and Industrial Wastes Association

Statler Hotel, Los Angeles, California

October 8–11, 1956

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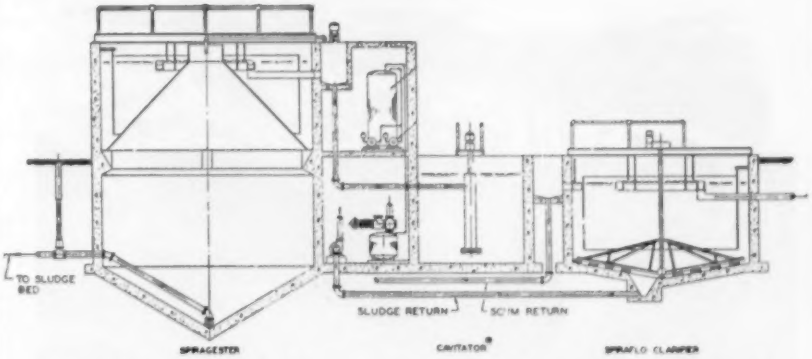

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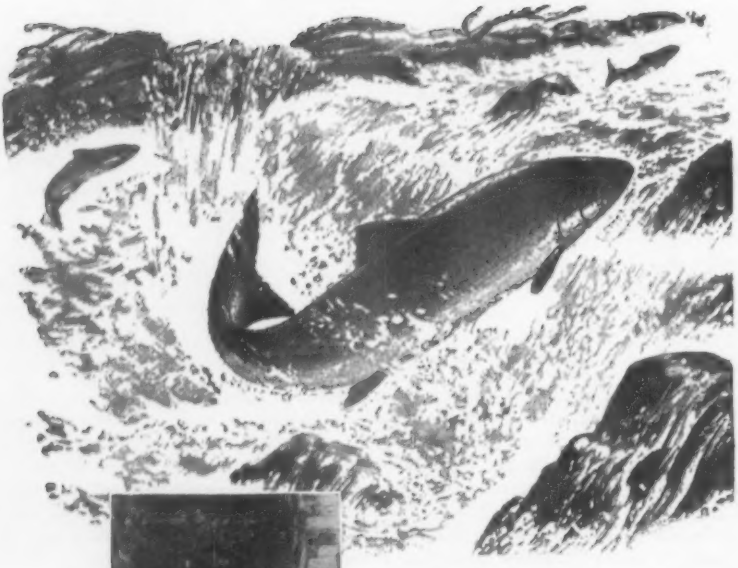

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After spawning the fish dies.

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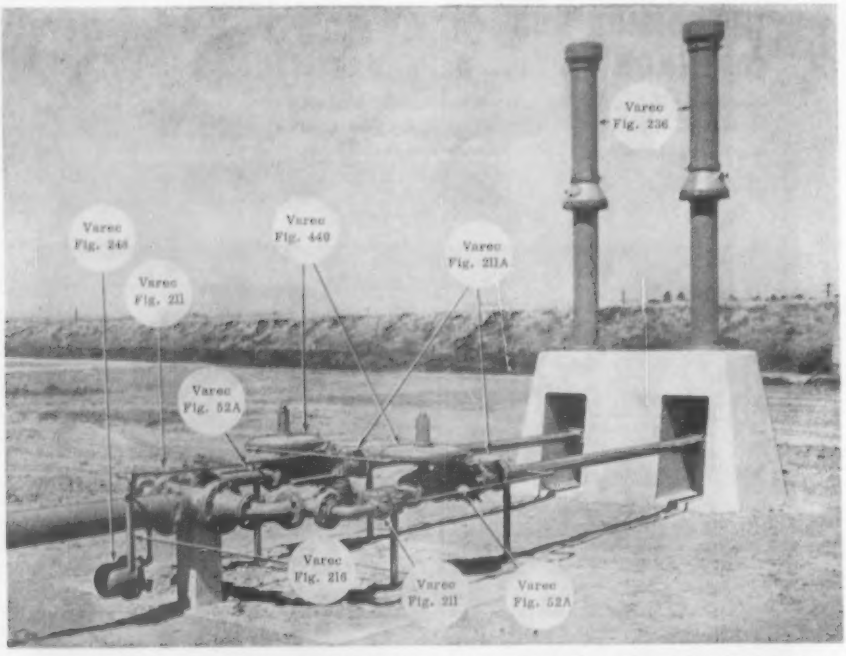

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(Typical Economies)

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- Savings in amount of material excavated
- Smaller earth loads
- Elimination of cradling
- Location of treatment plant at higher level
- Elimination of pumping

Users have found—With its high flow capacity (flow coefficient n=0.010 derived from the Manning formula) and with its maintained strength, Transite Sewer Pipe contributes to typical system design economies.

2. Installation

(Typical Economies)

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- Easier Handling
- Smaller crews
- Speedier assembly
- Assurance that joints pass severe infiltration tests

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(Typical Economies)

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(Typical Economies)

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- Sewers kept smooth and intact—pipe stays strong, joints stay resilient

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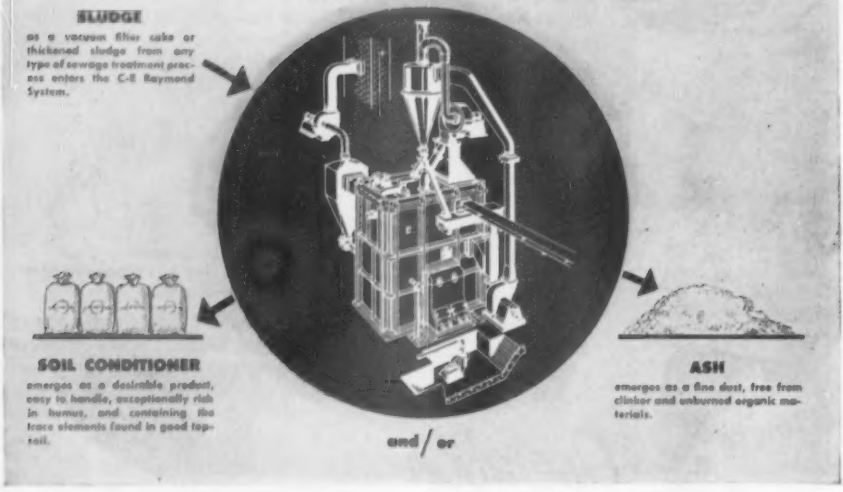
the asbestos-cement sewer pipe with the tight joints

JOHNS MANVILLE JM PRODUCT

The C-E Raymond System —

the MODERN method of SLUDGE DISPOSAL


SLUDGE
as a vacuum filter cake or thickened sludge from any type of sewage treatment process enters the C-E Raymond System.
SOIL CONDITIONER
emerges as a desirable product, easy to handle, exceptionally rich in humus, and containing the trace elements found in good top-soil.
and / or
ASH
emerges as a fine dust, free from clinker and unburned organic materials.

The C-E Raymond System is the most popular . . . most versatile on the market today. It combines the principles of Flash Drying and Incineration in a single compact system that allows you to Flash Dry or Incinerate sewage sludge alternately or concurrently. It is available with a high temperature deodorization system that assures odor-free operation.

Flash Drying by the C-E Raymond System is a continuous process in which vacuum filtered sludge — if uncontaminated with undesirable industrial wastes — is efficiently heat dried to a marketable soil conditioner retaining all its original nitrogen, but eliminating the objectionable qualities of the original wet sludge. In a well designed and operated "activated plant" the revenue from the sale of soil conditioner is sufficient to cover a sizable part of the plant operating costs.

Where the production of soil conditioner is not desired or the sludge is unsuitable, it is flash dried and then completely burned in suspension to an easily disposable, sterile ash.

The services of C-E specialists are available to help you plan for the end of atmospheric or stream pollution in your community, as has been done in so many others. For specific recommendations on the most practical answer to your sludge disposal problems, get in touch with the Combustion office nearest you. B-860A

also flash drying and incineration systems for industrial waste disposal

COMBUSTION ENGINEERING

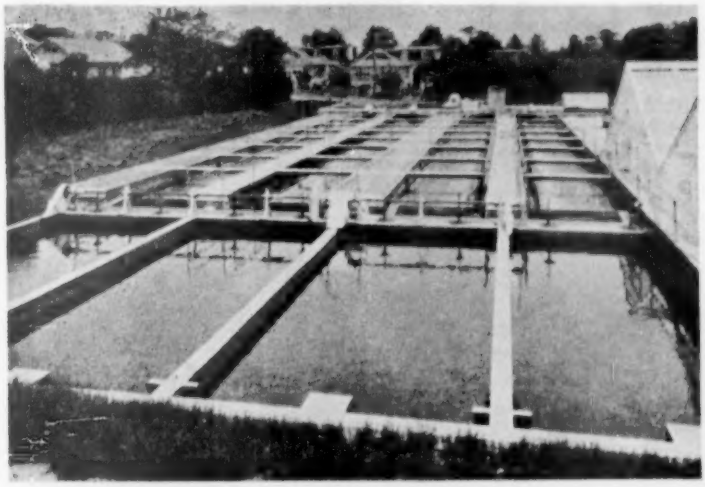
RAYMOND DIVISION

1315 North Branch Street, Chicago 22, Illinois

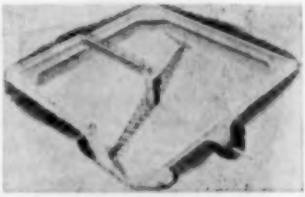
Eastern Office: 200 Madison Ave., New York 16, N. Y. *Western Office:* 510 W. Sixth St., Los Angeles 14, Calif.

CANADA: COMBUSTION ENGINEERING — SUPERHEATER, LTD., MONTREAL

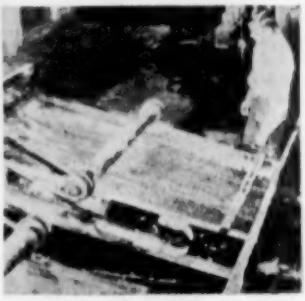
Call on LINK-BELT for water, sewage and waste treatment equipment built to last



STRAIGHTLINE COLLECTORS for rectangular settling tanks. Peak-cap bearings, pivoted flights and Straightline action all contribute to top efficiency. *Chain in system shown installed in 1930 gave 15 years of continuous 24 hour service before replacement.*



CIRCULINE COLLECTORS for square or round tanks for quick, positive sludge and scum removal in shortest time without septicity.



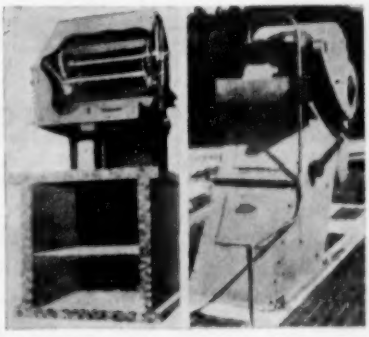
LIQUID SCREENS have proved efficient and economical for removing solids from industrial liquids. Available in several sizes with fine cloth.

ONE SOURCE . . . ONE RESPONSIBILITY

LONG life decides the eventual economy of sanitary waste treatment equipment. And after years of operation, Link-Belt systems in cities both large and small continue to provide highly effective water purification and sewage treatment service . . . with negligible maintenance expense.

Link-Belt equipment has also been welcomed in industrial plants of all kinds—particularly in states having strict anti-pollution legislation. For Link-Belt equipment can dispose of wastes at lowest cost—recover by-products for highest profit.

Link-Belt offers you a broad line of top-quality equipment . . . plus engineering proficiency acquired in over 34 years' experience. Our specialists will be glad to work with your own engineers, chemists and consultants to bring you the finest in modern treatment methods. Call your nearest Link-Belt office for full information and catalogs on the Link-Belt products shown here.



THRU-CLEAN BAR SCREENS — Remove large floating particles from large volumes of water — thus protect other equipment.

TRITOR SCREENS — For small sewage treatment plants—remove both grit and screenings with one mechanism.

CHEMICAL MIXERS — Link-Belt builds Straightline (shown) and flash mixers. Efficiency is assured by operation at variable speed for maximum floc formation.

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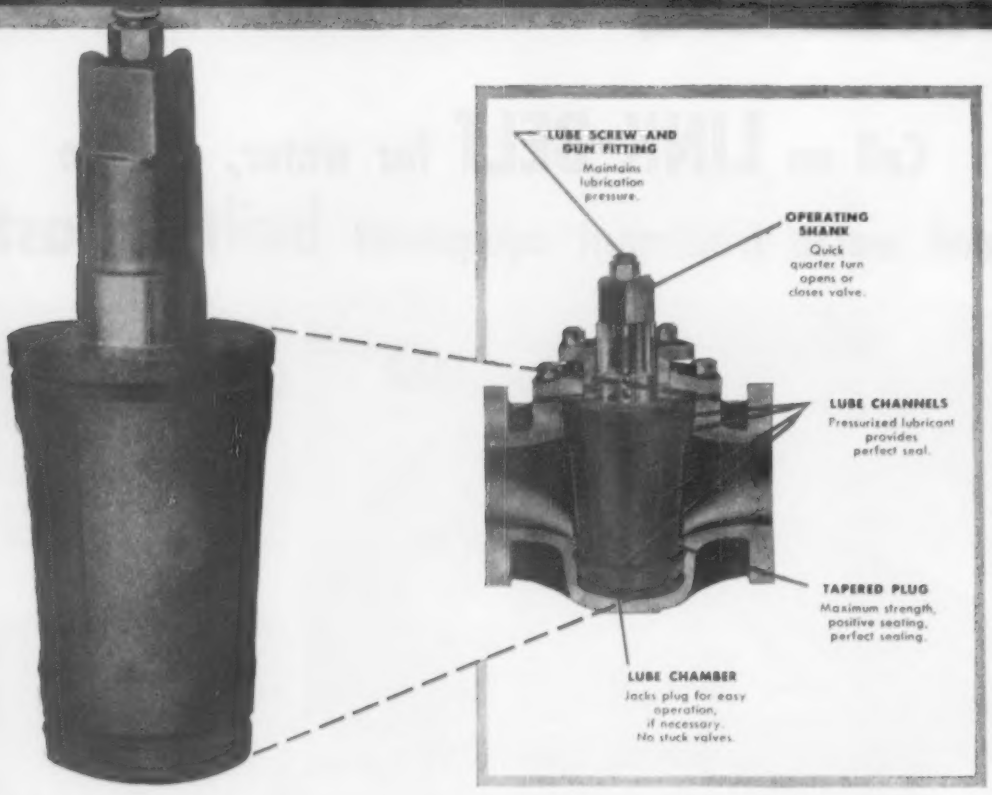
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= Trouble-Free Economy
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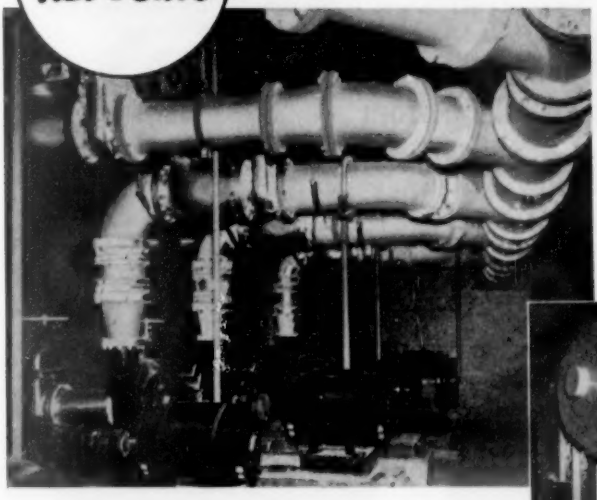
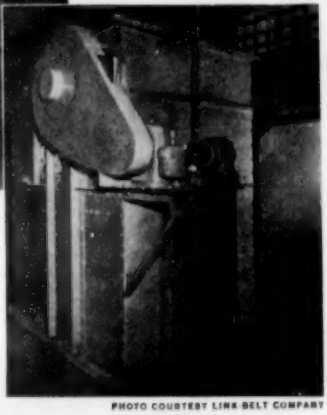


● **Pump Gallery:** Piping and equipment painted with *Glamortex® Enamel,* finest quality alkyd, in color, to assure a long-lasting, fade-resistant finish.



● **Walls and ceilings** throughout this modern plant, like those of the room housing the Link-Belt Grit Collector, are guarded with *Ramuc® Utility Enamel,* chlorinated natural rubber-base. Colorful *Ramuc* is easy to clean, chemical-resistant and won't saponify. It affords top maintenance economy as well as durable protection.

PHOTO COURTESY LINK BELT COMPANY

Specified by Engineers Toltz, King & Day, Inc., St. Paul, Minnesota, to fully protect this $2,000,000 plant

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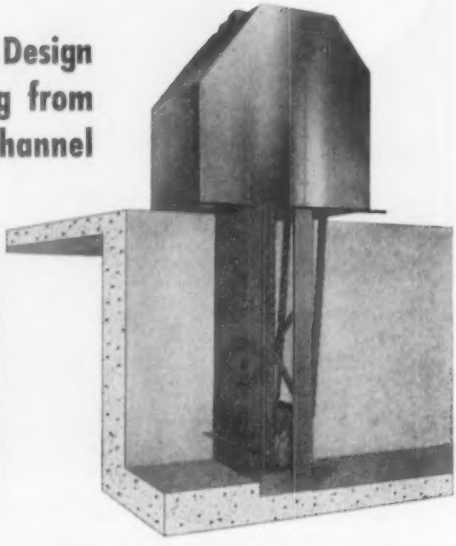
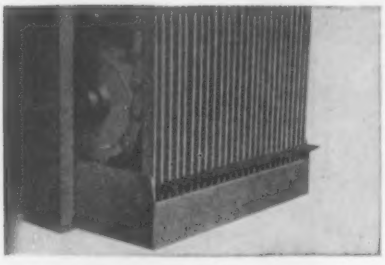
COMPLETELY AUTOMATIC MECHANICALLY CLEANED

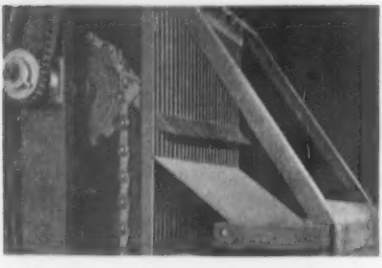
UNIT BAR SCREEN

Exclusive Pivoted Rake Design Assures Positive Raking from Absolute Bottom of Channel





Horizontal position of rake teeth at the beginning of cleaning cycle accomplishes raking from absolute bottom of channel. Gets under and picks up all screenings—does not push them away • Note unobstructed channel approach.



Rake making 45° radial drop to discharge screenings into trough—Note shear pin hub with tool steel shaft and hub inserts for perfect shear action.

THE "AMERICAN" UNIT SCREEN

Completely automatic with a limit switch which stops operation with the rake out of the flow when time control or water level control cycles have been completed • Easily installed as a unit in concrete channels • Total height of unit from mounting floor to motor base is only five feet • The entire above-floor structure is enclosed with stainless steel panels secured by quick opening fasteners to permit easy inspection and access.

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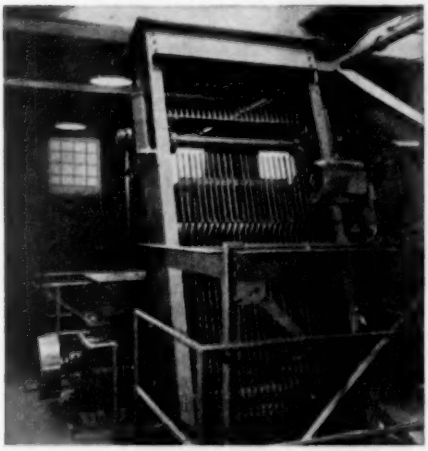

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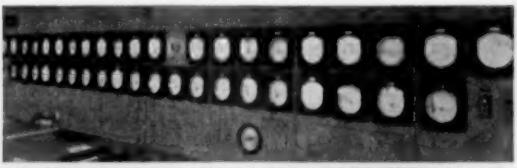

Penn Meters at Bryant Street Pumping Station Washington, D. C.

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In 1951, modernization of the Bryant Street Pumping Station was completed. This transformed one of the oldest pumping stations in Washington into one of the most modern and efficient stations in the country. The result was an increase of capacity from 89.5 to 310 million gallons per day.

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These meters provide an excellent example of matched metering at its best. The primary elements were all individually rated after installation and the meters were matched precisely to the rating curves by exact cutting of the calibration cam. This permits the highest reproducible accuracy which is a distinguishing feature of Penn meters.

In addition to the flow meters, Penn furnished all pressure recorders. These also are equipped with 10″ indicators.

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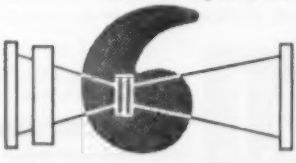

MATCHED METERING

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Behind control building, PFT Floating Cover on 35' digester

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WITH MODERN PFT EQUIPMENT

To meet its own rigid sanitary standards, the Springfield State Hospital at Sykesville, Md. constructed a complete sewage treatment plant in 1953. Key features of the plant are the items of PFT equipment installed for effective sludge digestion!

In the 35' digester, a *PFT Floating Cover* was specified for its positive scum submergence and safe utilization of gas. Floating Covers reduce explosion hazards by maintaining gas at positive pressure to prevent the entrance of atmospheric oxygen.

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SEWAGE AND INDUSTRIAL WASTES

Journal of the Federation of Sewage and Industrial Wastes Associations

Volume 27 **October, 1955** **Number 10**

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Sewage Works

IMPROVING THE EFFICIENCY OF EXISTING INTERCEPTORS

By Erik Riis-Carstensen

Associate Engineer, Buffalo Sewer Authority, Buffalo, N. Y.

Overflow Phenomena of Combined Sewers

As a basis for the search for means whereby the efficiency of intercepting sewers possibly can be improved at moderate cost, the present day knowledge of the various factors which have bearing on the subject has been reviewed.

The evaluation of the pollutional effect of overflows from a system of combined sewers eludes any available method of calculation because so many indeterminable factors are involved. In many localities, however, the effect is greater than would be expected, and the problem of abating pollution from this source has been given serious study by many cities. It has also been realized that any large city, served by combined sewers, would face enormous difficulties and probably prohibitive expenditures if required to eliminate the pollution caused by overflows.

Quantity

Calculations of frequencies, durations and quantities of sewage discharged thru overflows from a system of combined sewers have been made by many engineers in conjunction with reports on the design of intercepting sewers. The results of studies of the subject have been published by McKee (1), Hess and Manning (2), and Palmer (3).

Based of a thorough study of rainfall records from Buffalo, N. Y., made by Gascoigne in 1930 (4), the author prepared in 1934 a set of diagrams covering overflow data. The diagrams have recently been corrected to conform to the results of gagings made by the Buffalo Sewer Authority in the years 1938 to 1954 on numerous existing overflows. The corrected diagrams are shown in Figure 1.

It will be noted that a special term, the "Ch" or characteristic factor of a district has been used. The "Ch" factor indicates for a particular area the ratio of the average dry weather flow (g.p.d.) to the runoff (c.f.s.) during a uniform rainfall of one inch per hour. It is expressed as:

$$\text{Ch} = \frac{[\text{d}]\ \text{Ave. Per Capita Flow (g.p.d.)}}{\text{c}}$$

where "d" is the population density per acre and "c" is the runoff coefficient. This ratio is of great importance in the evaluation of excess capacity provided when the flow to be intercepted is determined in proportion to the dry weather flow.

Analysis of the actual gagings indicated that it would be reasonable to apply an allowance of plus or minus 30 per cent to the data shown in Figure 1.

Strength

If all sewers in a system were completely self-cleaning, and the entering storm water was pure, overflow would consist of normal sewage, diluted in proportion to the storm water carried. The annual quantity of sewage that

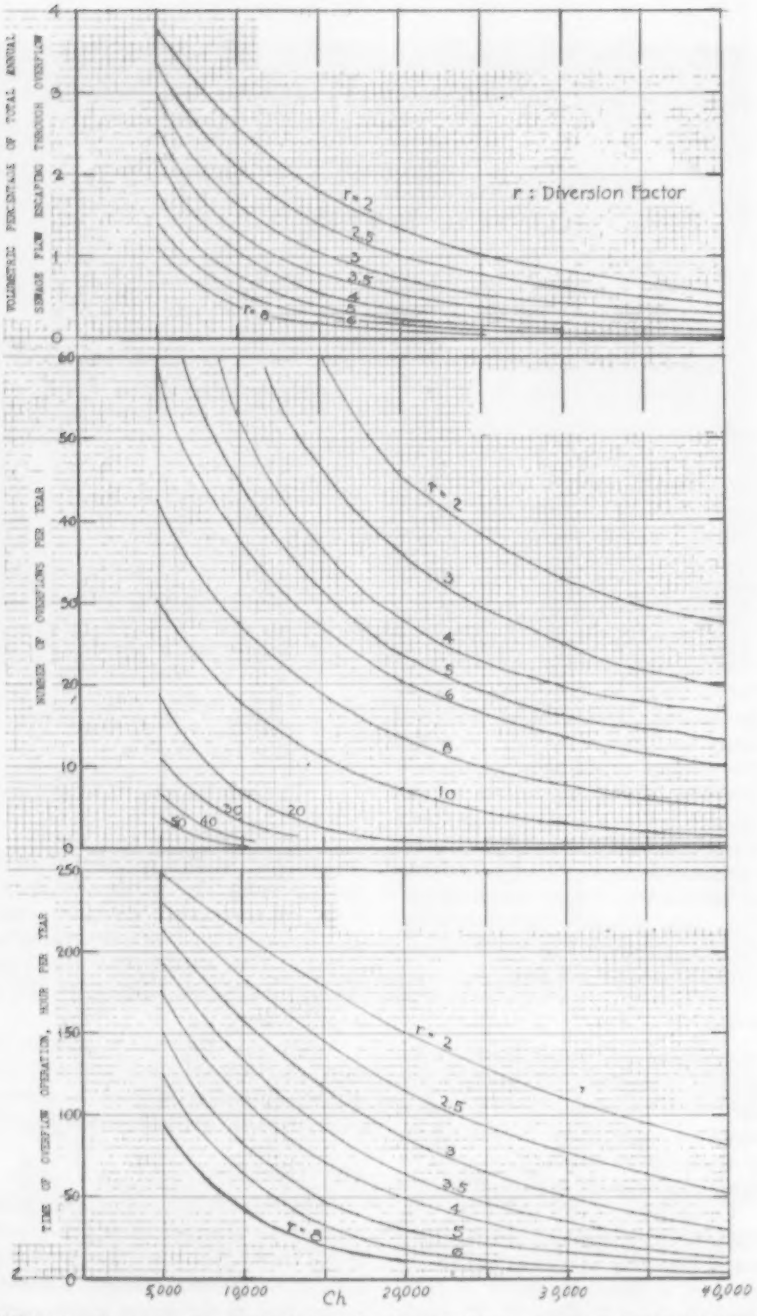

FIGURE 1.—Storm water overflow data from a combined sewer serving a district with variable characteristics.

would escape through the overflows could be estimated fairly well from Figure 1 or similar diagrams. It is probably true that these conditions are never fully met and that actual deviations vary greatly.

The entering storm water is never clean; a part of the dust and dirt collected on the streets will be washed into the sewers via the catchbasins. Palmer (3) reports, on studies made at Detroit to determine the quality of storm water, the samples of runoff taken from land surfaces were highly polluted. In addition the dry weather flow velocities are often so low that the sewers are not self-cleaning during rainless periods and deposits of sludge accumulate. This occurs more often in laterals than in trunk sewers.

During rain storms the deposits in catch basins and sewers are fully or partly stirred up and carried away by the flow. The process of scouring can be expected to continue some time after the storm has started and the dislodged material will move through the sewers with the velocity of the storm flow. In districts with long times of concentration, storm water from outlying areas will reach the trunk sewer outlet a long time after the beginning of the storm. In addition, the storm flow will include relatively clean water from areas where the sewers have already been washed out.

Other investigators have pointed this process out and report that the strength of the overflow water, after the capacity of the diversion to the interceptor of 2 to 2.5 times the dry weather flow has been exceeded, has the following characteristics: M.P.N. 4,300,000; 5-day B.O.D. 50 p.p.m.; suspended solids 250 p.p.m.; and suspended volatile solids 100 p.p.m. Erratic variations in the strength of the overflow are characteristic of storm water.

Under the direction of its consulting engineers the Buffalo Sewer Authority undertook an extensive program of gaging and sampling the sewage flow in 1936 and 1937 (5). Table I presents some of the results from these gagings made during the period August 5–22, 1936. Data from August 18, 19 and 20 were selected because a storm causing a precipitation of 0.55 in. fell on August 19 between 5:30 PM and 9:00 PM, while August 18 and 20 were rainless.

TABLE I.—Data from Gagings Made August 5–22, 1936

	Bird Avenue District				Bailey Avenue District			
	Av. 12 Rainless Weekdays	August			Av. 12 Rainless Weekdays	August		
		18[1]	19[2]	20[1]		18[1]	19[2]	20[1]
Flow (m.g.d.)	23.4	23.52	32.34	22.68	7.86	7.71	15.01	8.09
pH	7	6.9	7.1	7	7.3	7.1	7.5	7.3
Tot. solids (p.p.m.)	535	498	754	722	493	461	785	697
Tot. vol. solids (p.p.m.)	307	305	362	494	291	274	379	485
Susp. solids (p.p.m.)	186	158	544	178	117	126	436	52
Vol. susp. solids (p.p.m.)	132	108	350	104	81	92	196	42
Sett. solids, ½ hr. (ml./L.)	2.5	1.6	4.5	2	0.7	0.5	3	0.2
Sett. solids, 1 hr. (ml./L.)	2.9	1.6	4.5	2	1	1	3.5	0.4
Susp. solids (1,000 lb./day)	36.2	31	146.7	33.7	7.25	8.1	54.6	3.5
B.O.D., 5-day (p.p.m.)	164	162	100	111	119	127	121	72
B.O.D. (1,000 lb./day)	31.9	31.8	27	21	7.8	8.2	15.2	4.9
Chlorine demand (p.p.m.)	2.9	3.2	2.5	2.5	2.3	2.3	1.8	1.4
Chlorine demand (1,000 lb./day)	0.55	0.63	0.67	0.47	0.15	0.15	0.23	0.09

[1] No rain.
[2] 0.55 in., 5:30 PM to 9:00 PM.

TABLE II.—Flow and Suspended Solids, Bird Avenue District, August 19, 1936*

Time (P.M.)	Flow (m.g.d.)	Susp. Solids (p.p.m.)
5:30	25.3	208
6:30	120.6	964
7:30	104	1,220
8:30	56.7	696
9:30	37.6	364
10:30	28.6	172

Analysis

Period (P.M.)	Flow (m.g.d.)	Susp. Solids	
		1,000 lb.	Times Av. D.W. Rate
5:30–6:30	3.02	14.8	9.8
6:30–7:30	4.69	42.7	28.4
7:30–8:30	3.35	26.8	17.8
8:30–9:30	1.97	8.7	5.8
9:30–10:30	1.38	3.1	2.1
Total	14.41	96.1	12.8

* Reference (5).

Hourly samples were taken and composite samples were analyzed daily for total, volatile, suspended and settleable solids, B.O.D., and chlorine demand. The presence of large quantities of suspended solids, obviously caused by the storm flow, is noticeable in the August 19 results.

Table II shows the detailed gagings and samplings from the Bird Avenue District during the storm on August 19. The concentration of suspended solids reached 1,220 p.p.m., while the normal dry weather concentration was 186 p.p.m. Analysis of the data indicates that between 6:30 PM and 7:30 PM the suspended solids carried in the flow were 28.4 times the normal amount.

A series of overflows to a nearby drain are located upstream of a bottleneck in the Bird Avenue District trunk sewer. During the storm of August 19 probably 50 per cent of the flow was discharged through these overflows. If the full flow had reached the gaging point the amount of suspended solids carried between 6:30 PM and 7:30 PM would probably have been 50 or 60 times the normal dry weather flow. The total amount of B.O.D. in the Bird Avenue District sewer at this time was probably affected in a similar way by the upstream overflows. Comparison with the results obtained from the Bailey Avenue District where overflows did not exist supported this conclusion.

Fuller and McClintock (6) report the following composition of sewage at Macclesfield, England, during storm flows following a 0.1 in. rainfall on July 1, 1902 at the end of 8 days of dry weather:

Time (PM)	Times Dry Weather Flow	Susp. Solids (p.p.m.)
2:45	3	581
3:00	4	2,463
3:15	6	2,588
4:00	3	327

While the quantities and frequencies of overflow are fairly predictable when good precipitation records are available it is evident the strength of the overflow is subject to considerable and seemingly erratic variations. Any evaluation of the pollutional effect of an overflow may be grossly misleading if it is based only on volume.

It is quite possible large sludge deposits exist in many streams where overflows from combined sewers are discharged. These sludge banks exert a continuous oxygen demand on the stream waters for which other possible sources of pollution may be blamed. For this reason it may be valuable to have available a realistic appraisal of pollution due to overflows. However, to be reliable the appraisal should be based on the results of a series of gagings and samplings made for the particular case.

Improving Interceptor Efficiency

Financial limitations usually make it impossible for a large city served by combined sewers to abate pollution due to overflows. Changing to a system of separate sewers and constructing treat-

ment facilities would probably require greater expenditures than the taxpayers would approve. It is logical, therefore, to search for less costly means to improve the efficiency of existing interceptors and partially abate the pollution due to overflows.

The purpose of an interceptor is to carry the greatest possible amount of sewage solids, not necessarily flow, to the treatment plant. The efficiency of an interceptor should be measured by this yardstick. Consideration is not given here to the differences in capacity of a watercourse to absorb pollution at the various overflow locations.

Balancing the Diversion Factor

It is common practice to intercept flows in excess of the peak dry weather flow. In the usual method, diversion factors are chosen so flows up to a predetermined ratio of 2½ or 3 times the average dry weather flow will be intercepted. Another method of design provides for the interception of the average dry weather flow plus the runoff from rains of predetermined frequency.

The use of a predetermined ratio would appear to cause frequent undesirable overflows from districts where the "Ch" factor was small. The second method would cause excessive quantities of storm water to be intercepted. In an attempt to arrive at the best method, a study was made to determine which method would cause the least amount of pollutional matter to escape. Consideration also was given to the balancing of diversion factors so maximum efficiency could be obtained.

For illustrative purposes an imaginary sewer system was considered and a flow equal to three times the average dry weather flow was taken as the total amount of flow to be intercepted. All sewers were assumed to be self-cleaning and the diagrams in Figure 1 corrected accordingly. For simplicity all sewer districts were considered as having

TABLE III.—Comparative Estimates of Escaping Sewage Quantities Under Different Diversion Arrangements

Area (acre)		Arrangements						
		a	b			c		
		All Districts	Overflow Starts for Same Storm			Most Favorable Balance of r[1]		
Class A[2]	Class B[3]	Relative Amount of Escaping Sewage	Diversion Factor (r)		Relative Amount of Escaping Sewage	Diversion Factor (r)		Relative Amount of Escaping Sewage
			Class A	Class B		Class A	Class B	
500	1,000	100	5.24	2.7	101.8	3.75	2.9	99.2
1,000	1,000	100	4.8	2.52	100.2	3.75	2.8	97.9
1,000	500	100	4.28	2.32	99.6	3.47	2.7	96.6
500	1,000	100	5.24	2.7	110.1	3	3	100
1,000	1,000	100	4.8	2.52	112.9	2.63	3.1	99.9
1,000	500	100	4.28	2.32	114.7	2.63	3.2	99.6
500	1,000	100	5.24	2.7	90.6	5.25	2.7	90.6
1,000	1,000	100	4.8	2.52	87	4.88	2.5	87
1,000	500	100	4.28	2.32	86.5	4.31	2.3	86.5

[1] Diversion factor.
[2] Class A districts—d = 32 persons per acre, c = 0.4 and Ch = 8,000.
[3] Class B districts—d = 120 persons per acre, c = 0.6 and Ch = 20,000.

either a low or a high "Ch" factor. The time of concentration was assumed equal for all districts.

Three different conditions were studied:

1. The interception of three times the average dry weather flow.
2. The interception of the average dry weather flow together with the runoff from a storm of intensity "i" inches per hour.
3. The balancing of diversion factors to minimize the percentage of escaping sewage.

With this simplification calculation of the rainfall intensity to be used under the first condition is easy. Moreover, it is possible to determine by trial the "most favorable" combination of diversion factors.

The results of the study are shown in Table III. The Class "A" districts had an area (d) of 32 persons per acre and a runoff coefficient (c) of 0.4. The characteristic factor (Ch) was 8,000. In Class "B" districts "d" was 120 persons per acre, "c" was 0.6 and the "Ch" factor was 20,000. Indications are that the relative merits of alternate arrangements 1 and 2 depend upon the relative prevalence of districts with high and low characteristics. Fortunately all deviations from the ideal arrangement are insignificant in view of the factors of uncertainty involved; unless a marked difference exists in the strength of the sewage from the various districts.

The conclusion drawn from this study is that it is not possible to calculate a most favorable balancing of diversion factors for an actual sewer system even if the sewers were self-cleaning. A few calculations based on known data may be of help in formulating a decision, but the elaborate work required would not necessarily result in any actual improvement.

Use of Existing Capacities

To prevent surcharge of an intercepting sewer, regulating devices are ordinarily installed at the diversion chambers on the various individual combined sewers. In some cities the regulating devices are controlled by the water level in the interceptor. Other cities provide regulating devices designed to permit a predetermined volume of flow to enter the interceptor. Excess flow, regardless of the water level in the interceptor, is discharged through the overflow.

Storms, especially the numerous small ones, are rarely distributed evenly over the areas served by an extensive sewer system. Furthermore, the various sewer districts of the system have great differences in their times of concentration. Also some storage volume usually is available in the interceptor at the beginning of a storm. Consequently, the predetermined volume arrangement frequently results in some overflows operating long before the capacity of the interceptor has been reached. Frequent unnecessary overflows take place from one or more districts during minor local showers.

These unnecessary overflows often carry the greatest concentration of suspended solids. Where the interceptor controls are based only on the flow in the combined sewer, more efficient use of the intercepting sewer can be obtained at relatively moderate costs by modification of the existing intercepting arrangements. The provision of supplemental devices would permit interception as long as the hydraulic gradient in the interceptor was below a predetermined level. Supplemental devices would naturally be most beneficial where the amounts of suspended solids carried by the sewers during the early stages of storm flow are particularly high. In addition, they would provide a reserve connection to the interceptor in case of clogging of the original connection.

A rational calculation cannot be made of the benefits obtained by an interceptor controlled sewer system unless extensive long term data are avail-

able on the volume and strength of simultaneous flows.

Stand-by Tanks

The storage capacity available in a large intercepting sewer at the beginning of a storm has some value when utilized for the collection of the storm water which carries a heavy load of suspended solids. Flexible interceptor controls may entirely eliminate overflow from a large number of the many minor storms. It is obvious that the volume of flow containing suspended solids during intense storms far exceeds the amount that can be carried away or stored in the intercepting sewers.

If a high degree of overflow elimination is required for pollution abatement reduction can be obtained by the construction of stand-by tanks. These tanks may serve individual districts or be constructed at convenient points along the intercepting sewers, thus adding general storage capacity. Where the individual sewer districts are large and the times of concentration long, enormous capacities would be required if the tanks intercepted a substantial amount of the solids laden storm flow.

It is possible, however, that the retention principle can be utilized at moderate expense when designs are made to conform closer to what may be called the "logistics of suspended solids deposits." Study of strengths of overflows reveal that the shorter the time of concentration, the greater and more sharply defined is the strength of the overflow. During a storm the flow in the combined sewer will increase rapidly. Deposits collected during the preceding dry weather period will be dislodged and carried in the sewers with approximately the same velocity as the storm flow.

In a lateral sewer this scouring process will occur for only a short time before the storm flow will change character and become quite free from street and sewage solids. Depending on the time of concentration, storm water from washed and unwashed sewers will become more and more mixed as it travels down the sewer. The stand-by tank capacities required to retain a substantial part of this flow will increase with the size of the tributary district. Therefore, stand-by tanks become less and less efficient with longer times of concentration. In principle, a system of small local tanks properly distributed over a district would be far superior to a single large tank at the outlet.

While the thought of a multiple system of storm water tanks is rather overwhelming at first, several factors are frequently present to make possible worthwhile improvements by such arrangements. It is common in the small branch and lateral sewers that the dry weather flow velocities are too low to prevent the formation of deposits. Areas with sewers that collect deposits are the principal sources for the heavy loads of suspended solids carried by storm flows. Furthermore, it is feasible to construct local storm water tanks which would be by-passed by the dry weather flow. These tanks would collect and retain a predetermined volume of storm flow and automatically discharge this water to the trunk sewer when normal dry weather flow has been established.

In many localities a well planned system of local storm water tanks would materially reduce the amount of suspended solids which otherwise would be discharged through the overflows. This plan offers the advantage of progressive construction, permitting units to be built as needed.

Sewer Reconstruction

One way to minimize the discharge of suspended solids through the overflows would be a program of reconstructing the sewers that collect deposits. Such a program might require expenditures not justified by the results obtained. However, it does appear to merit consideration, particularly in connection

with planning for the future or where reconstruction is needed anyway.

Dry weather flow velocities have apparently been given more serious consideration in European than American sewer practice. Copenhagen, Denmark requires that any combined sewer shall be designed so the velocity of flow can clean the sewer at least once each day. The maximum flow on the week day with the smallest flow must produce a velocity of 2 ft. per sec. in the large sewers, 2.5 ft. per sec. in the smaller sewers and 3.3 ft. per sec. in the 8-in. sewers.

To facilitate sewer construction with a minimum of pumping, wide use is made of circular sewers with elliptical inverts. These pipes are ordinarily made of vitrified clay or concrete and cost slightly more than circular pipes.

Sewer Flushing

The flushing of sewers having grades too flat for self-cleaning is an old and well known practice, and cannot be omitted when analyzing means for improving the efficiency of interceptors. Regular flushing of the sewer deposits would tend to minimize the discharge of suspended solids through the overflows.

To be effective flushing must wash the deposits to downstream sections where the velocity is adequate. A schedule of simultaneous street flushing extending over entire sewer subdistricts might prove particularly effective. The most advantageous arrangement would depend entirely upon local conditions.

Summary

A number of methods are available by which pollution caused by storm overflow from combined sewers can be reduced.

Wherever a need for pollution abatement exists or is anticipated in the future, the first step should consist of a direct survey of existing conditions. This survey would include a series of gagings and samplings of storm overflow and storm flows at selected points throughout the system. A comprehensive analysis of dry weather flow velocities in all sewers, particularly in branch and lateral sewers is necessary. On the basis of a conscientiously executed survey a progressive plan of improvements could be drafted, subject to the limits dictated by sound economics. The plan would be directed toward abating pollution as effectively as possible.

Although the year to year benefits may not be apparent, the existence of such a program might easily make a measurable difference over a period of 50 yr.

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MECHANICS OF OXYGEN ABSORPTION IN SPIRAL FLOW AERATION TANKS

III. Application of Formulas

By Henry R. King

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Formulas for computing oxygen absorption rates in spiral flow aeration tanks have been derived and checked experimentally. Their usefulness, however, depends upon what their applications will be.

Their greatest value probably will not result from the aid they give in estimating air requirements, but rather from the fact that they relate all the factors affecting the entrance of oxygen into solution. For this reason, they should be helpful in the design and operation of activated sludge plants, because they serve as a basis upon which better analyses can be made of the many problems encountered in the design and operation of the aeration tanks. Sanitary engineers will at once see ways in which the formulas will be useful. However, a few illustrations may be of interest.

Computed Air Requirements

At present, the formulas cannot be used with any degree of certainty for computing the air requirements for any particular aeration process. Too little is known as yet regarding the extent to which sewage solids are oxidized in the aeration process as related to the value of the oxygen absorption coefficient and the quality of the effluent. Also, additional data are required concerning the size of air bubbles under operating conditions. Air consumption estimates on the basis of air necessary per pound of B.O.D. removed, as indicated by experience, is a more practical method. The formulas, however, should serve as a guide in aeration plant design where the object is to keep the costs of power for the compression of air as low as economically practical.

Power Requirements

The oxygen absorption rate and power per pound of oxygen absorption are shown in relation to rate of air application in Figure 10 for the aeration of clean water at 20° C. and zero D.O. with air diffusing at 15-ft. depth. Four conditions of air diffusion are included; that is, 5 and 10 per cent diffuser plate areas with air bubbles of 0.1 and 0.15-in. diameter. The diagram is based on Eqs. 6 and 7. At 10° C. and the same conditions otherwise, the absorption plates would be slightly lower and the power requirements slightly higher than at 20° C. The power requirements are gross. Allowances have been made for blower and motor efficiencies and piping and diffuser losses.

In the activated sludge process, the power requirements per pound of oxygen absorption are much higher than shown in Figure 10. This is due to the oxygen absorption coefficient being lower than in clean water, and also due to the fact that in plant operation several p.p.m. of D.O. must be maintained in the mixed liquor to insure aerobic conditions. It is probable that for the same rate of air application the rate of oxygen absorption in the conventional activated sludge process averages only about 60 per cent of that shown in Figure 10 and, consequently, the power must be about 167 per cent as much.

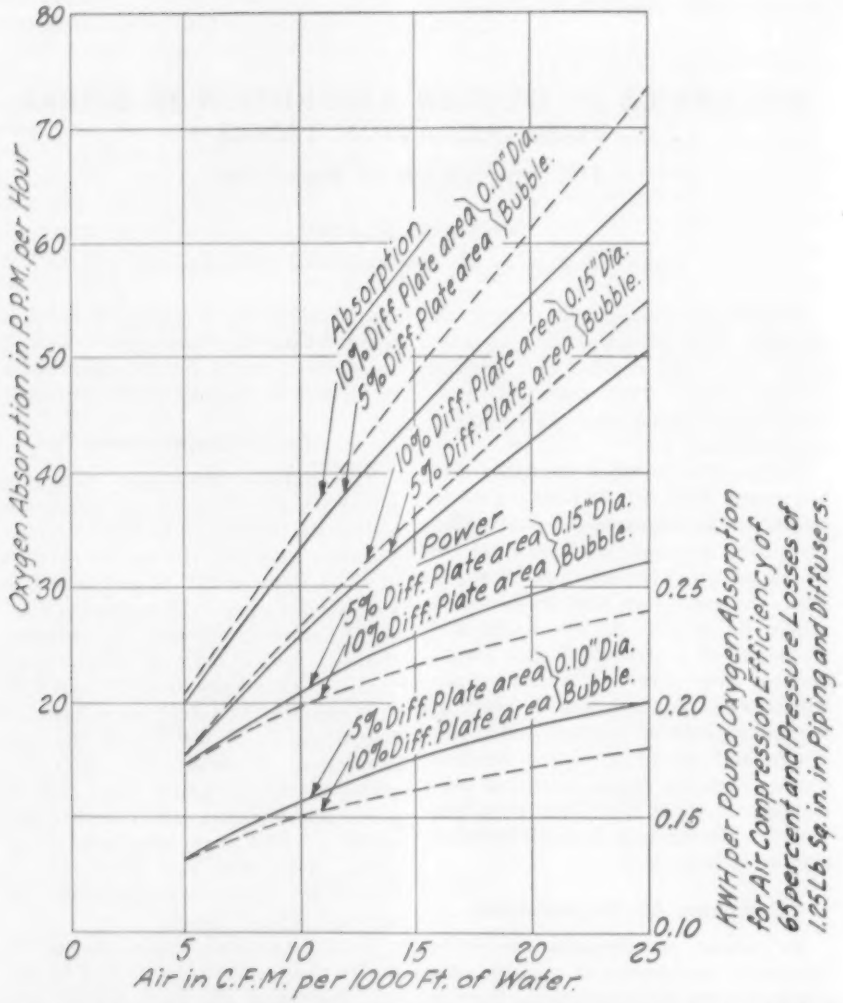

FIGURE 10.—Oxygen absorption rates and power requirements in aeration of clean water in 15-ft. depth spiral flow tank. (Based on Eqs. 6 and 7; water at 20° C. and zero D.O., barometer at 29.3 in. Hg.)

To relate power per pound of oxygen absorption and power per pound of 5-day B.O.D. removal requires some knowledge of the extent to which the B.O.D. removed from the sewage is oxidized in the aeration process. Tests by Heukelekian (18) indicated that a 75 per cent reduction in the B.O.D. removed by process produced a good sludge and effluent in the conventional activated sludge process.

On this basis, 1.0 lb. of 5-day B.O.D. 75 per cent oxidized would require for the total B.O.D. oxidized about $1.5 \times 1.0 \times 0.75$, or 1.12 lb. of oxygen. Therefore, the power per pound of 5-day B.O.D. removal for such an extent of oxidation in the process would be, on

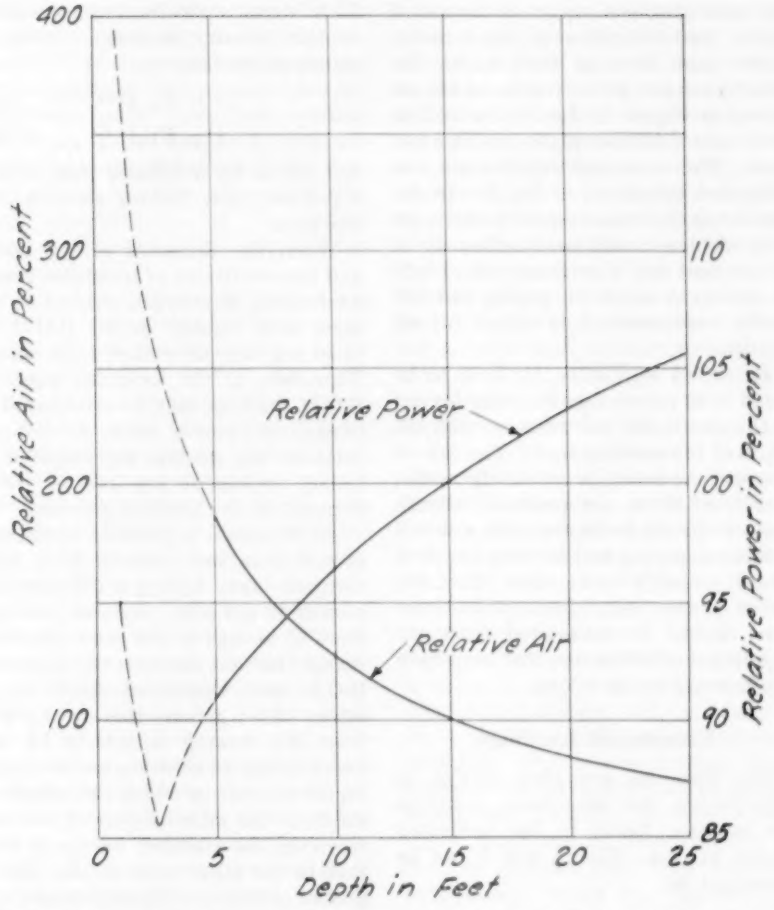

FIGURE 11.—Air and power requirements in relation to depth in spiral flow aeration tanks. (Diffuser plate area 5 per cent, air bubble diameter 0.13 in., water at 20° C. and 7 p.p.m. oxygen deficiency or 10° C. and 9 p.p.m. oxygen deficiency, air compression efficiency 65 per cent over-all, air piping and diffusion losses 1.25 lb. per sq. in.)

the basis of the foregoing, about 167 × 1.12, or about 187 per cent of that per pound of oxygen absorption for clean water at zero D.O. (Figure 10), for any particular rate of air application.

It should be noted that Figure 10 shows that as the rate of air application increases, the power per pound of absorption also increases. Therefore, the greater the B.O.D. loading of any particular aeration tank, all other conditions remaining unchanged, the more the power per pound of oxygen absorption accomplished.

Economical Tank Depth

The formulas may be used to determine the economical depth of aeration tanks. This can be done easily when the aeration channels of various depths are geometrically similar. For example, for the same conditions of sewage treatment—that is the same bubble size,

the same dissolved oxygen in the mixed liquor, and detention in the aeration tanks—and identical final tanks, the relative air and power requirements are shown in Figure 11 for depths of 5 to 25 ft. and a diffuser plate area of 5 per cent. The curve for relative air was computed by means of Eq. 6. In determining the relative power curve, an over-all blower and motor efficiency of 65 per cent and a pressure loss of 1.25 lb. per sq. in. in the air piping and diffusers were assumed to obtain for all depths.

Referring to Figure 11, it is to be noted that power requirements for air in this particular case increase with the depth of the aeration tank. The rate of increase, however, is relatively small; therefore, there are probably advantages in deeper tanks since the reduced tank area, piping and blowers required would probably more than offset the added power costs. Each plant, however, should be considered from the standpoint of all factors that may have any bearing on its design.

Economical Loadings

The formulas are also helpful in determining the economical loadings for aeration tanks in the activated sludge process. Eqs. 6 and 7 can be converted to

$$R = K_1 (A)^{H^{0.05}/0.85} \ldots . (15a)$$

$$R = K_2 (A)^{H^{0.04}/0.90} \ldots . (15b)$$

respectively, in which K_1 and K_2 are constants obtaining when all factors remain unchanged except the air application rate, R, and oxygen absorption rate, A. By dividing the right side of Eqs. 15*a* and 15*b* by A, the following forms result

$$r = K_1 (A)^{H^{0.05}/0.85-1} \ldots . (16a)$$

$$r = K_2 (A)^{H^{0.04}/0.90-1} \ldots . (16b)$$

in which r is the air application rate in c.f.m. per 1,000 cu. ft. per p.p.m. oxygen absorption per hour. For the 15-ft. depth tanks in most general use in this country in large plants, the equations become

$$r = K_1 A^{0.35} \ldots . (17a)$$

$$r = K_2 A^{0.24} \ldots . (17b)$$

Eq. 17*a* is for a diffuser plate area of 5 per cent, Eq. 17*b* for an area of 10 per cent.

When the character of the sewage and the conditions of treatment remain unchanged, the oxygen absorption rate must vary directly as the B.O.D. applied per day per unit of tank volume. Therefore, in the foregoing equations B.O.D. loadings may be substituted for oxygen absorption rates, in order to compute the relative air required for sewage treatment per unit of B.O.D. removal by the aeration process.

To illustrate a possible application of such equations, consider 15-ft. depth aeration tanks having a diffuser plate area of 10 per cent. Assume two equal flows of sewage of the same character, except that one contains twice as much B.O.D. and suspended solids as the other. Also, assume these two sewage flows are treated separately by activated sludge in aeration tanks of equal liquid volume, in which the suspended solids in the mixed liquor of the tank receiving the stronger sewage is twice that in the other tank, so that the degree of oxidation of the activated sludge remains identical in both.

Then the air required per pound of B.O.D. removed from the stronger sewage would be $(2)^{0.24}$, or 1.18 times that necessary for the weaker sewage. This extra air could be reduced if more aeration tank volume were used for the stronger sewage, the economical addition, however, depending on the relation of tank cost and power for air compression. The extra air could also be reduced by using more than a 10 per cent diffuser area for the stronger sewage.

No study of aeration tank loadings would be complete, however, without also considering the related require-

ments for final settling tanks. In the foregoing illustration, the higher concentration of suspended solids in the mixed liquor for the stronger sewage would also mean more final settling tanks if an effluent were to be obtained equivalent to that from the weaker sewage and also in order to produce adequate concentration of the return sludge without any longer period away from aeration.

Eqs. 17*a* and 17*b* can be used in another way for determining the economical B.O.D. loading for aeration tanks. If, for example, a sewage has been treated in an activated sludge process for sufficient time to establish the air requirements per pound of B.O.D. applied, the economic loading for sewage of similar origin may be explored. As long as operating conditions remain the same (that is, the same sewage temperature, diffuser plate area ratio, bubble size, and ratio of total suspended solids in the mixed liquor to that entering in the sewage in an hour or day), the equations can be used to determine the relative air requirements for various B.O.D. loadings. By adding the annual costs for compressed air to those for aeration and final tank volumes required, the most economical aeration tank loading may be determined. However, some knowledge of the characteristics of final settling tanks is necessary in order to properly relate their requirements to the solids concentrations in the mixed liquor.

Sludge Reaeration

Another analysis by means of the Eqs. 17*a* and 17*b* may be illustrated in considering sludge reaeration as a means of reducing the volume of aeration tank required. For example, assume that as a substitute for the aeration tank of a conventional activated sludge process, the first half is to be retained for mixed liquor aeration and only one-fourth of the second half for the aeration of the return sludge, with a suspended solids concentration four times that in the mixed liquor. Thus only five-eighths as much tank volume would be used for the same degree of oxidation of the activated sludge, since the solids under aeration would remain the same in either case. However, to maintain D.O. in the sludge reaeration equivalent to that in the second half of the original aeration tank, for a diffuser plate area of 10 per cent, would require $(4)^{0.24}$, or 1.4 times as much air per unit of oxygen absorption. Thus, the smaller tank volume with sludge reaeration would require for the total aeration period an average of 1.2 times the air necessary for the original tank.

Also, it is quite probable that the effluent in the case of this arrangement for sludge reaeration would be inferior because the sewage would be in contact with the activated sludge for just one-half the time it would in the conventional activated sludge process, before entering the final tanks. A decision on the use of sludge reaeration would have to be based on the costs of compressed air and tank volume, with consideration given to the over-all sewage treatment efficiency. Use of considerably larger diffuser area for sludge reaeration than is normally used for activated sludge might bring the air requirements for the two systems very close together, particularly where the diffusers are such as to produce a relatively small bubble size.

Aeration Tank Volume

Eqs. 17*a* and 17*b* also may be applied in studying the volume of aeration tank required for an activated sludge process. Assume that two different sizes of 15-ft. depth aeration tanks are being considered, one four times as large as the other. Also assume that the smaller tank would operate with four times the concentration of suspended solids so that the solids under aeration would be the same in either case, thus assuring the same degree of oxidation of the ac-

tivated sludge no matter which tank is selected. The air required for the smaller tank for a 10 per cent diffuser plate area would be $(4)^{0.24}$, about 1.4 times that for the larger. (The additional air required could be reduced, however, by using more plate area.) In addition to the added power for air, in case the smaller aeration tank were used, there would be other additional costs such as more final settling tanks and a greater rate of sludge return if an equivalent effluent were to be produced. Thus, there are economical limitations as to the extent to which aeration tank volume can be reduced by increasing the concentration of solids in the mixed liquor. Also, there are other limitations—such as, for example, the difficulties of producing in the final tanks a return sludge of solids concentration beyond a certain point.

Research

In addition to the value of the equations for analyses of design and operation problems, they also are useful for research work in the laboratory. It is an advantage to be able to convert the results obtained in a small tank to a plant-size installation. The way is opened for tests on a small scale to improve aeration efficiency.

For example, the deflecting baffles in the 3-ft. depth experimental tank were removable. The rate of oxygen absorption was the same with and without these baffles. Evidently, the lowered velocity of overturn with the baffles removed resulted in additional oxygen absorption from the bubbles, because of their longer detention, just compensating for the reduced absorption due to the lower surface velocity. Such a test raises the question as to the value of deflecting baffles in all plants. When the rate of air application is low, baffling may be necessary to provide adequate mixing.

The formulas as developed apply to porous plate air diffusers. Nevertheless, other types of diffusers can be compared with the porous plates in the laboratory. It is quite probable that the same relative efficiencies obtained in small experimental tanks would hold for plant-size installations.

Summary

Formulas have been presented for computing the rates of oxygen absorption in spiral flow aeration tanks in which air diffusion is by means of porous plate diffusers. They are based on published data pertaining to the rates of oxygen absorption by water from atmospheric reaeration and rising air bubbles, and on the results of tests made in the Sanitary District of Chicago on circulating velocities in large aeration tanks. By means of the hydraulic model theory, formulas applicable to any tank depth were derived. These were checked by means of laboratory tests on small tanks and also on large aeration tanks. Determinations were made of the effects of such factors as air application rate, air bubble diameter, oxygen deficiency, water temperature, and pollution.

The formulas show the relationship of the various factors affecting oxygen absorption in aeration tanks. Thus they are helpful for research purposes and for use in connection with problems involved in the design and operation of the activated sludge process.

Acknowledgments

This series of papers was made possible through the cooperation and help of associates in the Sanitary District of Chicago. The author is particularly indebted to Norval E. Anderson, Engineer of Treatment Plant Design, in whose department the work was done, and also to John R. Palmer and E. H. Morgan, Engineer of Plant Operation and Principal Chemist, respectively, of the North Side sewage treatment works, where the tests were made.

W. A. Dundas is General Superin-

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Errata

In the computation for Table I of Part I of this series the bubble diameter varies as the bubble volume raised to the ⅓ instead of the ⅔ power. Correctly computed the oxygen absorption rates in Table I are about 2 percent lower than those shown.

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RESEARCH ON ACTIVATED SLUDGE

VI. Oxygenation of Water with Air Bubbles

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Since the publication in 1924 by Lewis and Whitman of their "Principles of Gas Absorption" (1) these observations have found widespread attention and have finally come to be almost generally accepted as the "two-film theory" or the "liquid film theory."

Danckwerts (2) made the following observation on this subject: "The fictitious nature of the 'liquid film' is probably widely expected; nevertheless it is constantly referred to as though it really existed. . . . The present theory of gas absorption is based on a picture of the absorption mechanism which is probably unrealistic. For many practical purposes this is unimportant, but the theory is likely to be misleading when used to predict the behaviour of systems for which no experimental data are available."

It is true that the "liquid film" concept does not throw much light upon what actually happens at the interfacial surface. The application of this theory sometimes leads, even in simple cases, to a totally incorrect idea of the nature of the diffusion process.

Lewis and Whitman based their principles upon the relation:

$$\frac{dW}{A\,dt} = K_L\,(c_{L_s} - c_L)$$

where K_L is the "diffusion coefficient for liquid film."

This relation may be of use in a number of practical cases. In principle it means the velocity of solution is proportional to the deficit. It is not possible on this basis to build up theoretical considerations as to what takes place at the interfacial surface, as the formula is in insufficient agreement with Fick's diffusion equation which is of general application:

$$dm = A\,k\,\frac{dc}{dx}\,dt$$

This relation indicates that the amount of matter diffused during the very short time dt is determined by the area of the cross-section A, the diffusion constant k and the concentration gradient $\frac{dc}{dx}$ ruling on either side of the cross section.

The diffusion constant k is a physical constant, the numerical value of which is equal to that of the amount of matter which diffuses through unit area in unit time when the unit concentration gradient exists on either side of the interfacial surface.

The diffusion equation for one particular moment may be formulated as:

$$Q = A\,k\,\frac{c_1 - c_2}{h}\,t$$

in which $(c_1 - c_2)$ is the difference in concentration over a distance h, in order to indicate the relation existing at that moment between the values that determine how much matter (gas) becomes diffused. However, it must always be remembered that $\frac{c_1 - c_2}{h} = \frac{dc}{dx}$ is the concentration gradient which applies during the very brief moment dt and that this concentration gradient is a value that varies with time.

In other words, the rate of diffusion

at an interfacial surface is determined by the concentration *gradient*, the value of which depends upon the time of existence of the interfacial surface, and not merely by the *difference* in concentration.

The relation used by Lewis and Whitman, which does not take this fact into account, therefore leads to serious inaccuracies in the conception of what occurs at the interfacial surface.

A consideration of fundamental importance is that the "resistance" which controls the rate of diffusion is not primarily due to the existence of thinner or thicker films of gas or liquid, but to the fact that the rate of diffusion is inversely proportional to the square root of the time of existence of the interfacial surface. This velocity is therefore a maximum at the moment of formation of the interfacial surface and afterwards decreases very rapidly with the time.

Danckwerts (2) gives a mathematical exposition of gas absorption under a variety of conditions, taking as a basis Fick's law of diffusion and entirely abandoning the "stagnant film" theory.

The following study is based on this same principle, although the procedure is different from that of Danckwerts. The study is limited to a single case, *viz.*, the dissolving of oxygen from an ascending air bubble in water. The correctness of the theoretical procedure has been confirmed by experiment.

Theoretical Considerations

During the ascent of an air bubble through water of low oxygen content, oxygen passes by diffusion from the air into the water. The rate at which this proceeds depends to a considerable extent upon the rate at which the layers of water molecules adjacent to the air bubble are replaced by other layers. The continuous variation of boundary layers which occurs in reality cannot be approximated by a mathematical expression. This variation of the boundary layers may be considered to be replaced by a variation of a different kind in which the boundary layers remain unchanged during a certain time, after which they suddenly become mixed with the remainder of the liquid and are replaced by other layers. Use is made of the following additional symbols or supplementary definitions:

A = area of boundary (sq. cm.);
c = concentration of oxygen (g./ml.);
c_{atm} = concentration of oxygen in water at a pressure of one atmosphere of oxygen;
c_L = concentration of oxygen in water;
c_{Ls} = concentration of oxygen in water at saturation;
d = diameter of air bubble (cm.);
$f = \frac{Q_{O_2}}{V_{O_2}}$ (g./ml.);
h = height of water column through which the air bubble has passed (cm.);
h' = height of water column over which the water layers in the boundary of the air bubble are considered to remain quiescent;
k = coefficient of diffusion of oxygen in water (sq. cm./sec.), assumed in this research to be 2×10^{-5} sq. cm./sec.;
$n = h/h'$;
p = pressure in air bubble (atm.);
Q_d = quantity of diffused oxygen (g.);
Q_{O_2} = quantity of oxygen in air bubble when $p = 1$ atm. (g.);
t = time of contact (sec.);
t' = time of existence of hypothetical interfacial surface at rest $= h'/v$ (sec.);
v = velocity of ascent of air bubble in water (cm./sec.);
V_b = volume of air bubble (ml.);
V_{O_2} = volume of oxygen in air bubble;
V_{O_2h} = volume of oxygen in air bubble after distance h;
$V_{O_2h'}$ = volume of oxygen in air bubble after distance h';

V_{dh} = volume of oxygen diffused from air bubble after distance h, expressed as a percentage;

$V_{dh'}$ = volume of oxygen diffused from air bubble after distance h';

$\frac{1-u}{\gamma}(c_{Ls}-c_L)$ = absolute concentration gradient, where $(c_{Ls}-c_L)$ = a concentration of oxygen on one side and $u(c_{Ls}-c_L)$ = a concentration on the other side of a water layer with a thickness of γ cm.;

$\chi = \frac{1-u}{\gamma}$ = relative concentration gradient, or the concentration gradient where $c_{Ls} = (1+a)$ and $c_L = a$.

During the period of existence of the interfacial surface air/water at rest, the rate at which the oxygen passes into the water (with 0.0 mg. O_2 per l.) is expressed by the relation:

$$Q_d = 2\,A\,c_{Ls}\sqrt{\frac{kt'}{\pi}} \quad \ldots\ldots (1)$$

c_{Ls} may then be formulated as: $p\frac{V_{O_2}}{V_b}c_{atm}$, and $\frac{dQ_d}{dt}$ as:

$$-\frac{dV_{O_2}}{dt}\,f\,p$$

Eq. 1 then becomes:

$$-\frac{dV_{O_2}}{dt} = \frac{1}{f}2\,A\,\frac{V_{O_2}}{V_b}c_{atm}\sqrt{\frac{k}{\pi}}\frac{1}{2}t'^{-\frac{1}{2}}$$

from which can be deduced on integration

$$V_{O_2h'} = e^{-x} \quad \ldots\ldots (2)$$

in which

$$x = 2\,\frac{A}{V_b}c_{atm}\,\frac{1}{f}\sqrt{\frac{kt'}{\pi}}$$

Since $V_{O_2h} = (V_{O_2h'})^n$ it follows that

$$V_{dh} = 1 - (e^{-x})^n \quad \ldots\ldots (3)$$

$$-\ln(1-V_{dh}) = \frac{h}{h'}2\,\frac{A}{V_b}\frac{1}{f} \times c_{atm}\sqrt{\frac{k}{\pi}}\sqrt{\frac{h'}{v}} \quad \ldots (4)$$

It has been assumed in this deduction that the oxygen content of the water in which the air bubble ascends is invariably equal to zero.

There are certain conditions which have not been taken into account in the deduction. First there is the increase in volume of the air bubble as a result of decrease in pressure during ascent and the decrease in volume due to dissolution of oxygen from the air bubble. It can be ascertained by calculation that in the case of a 3 m. column of water and $V_{dh} = 50$ per cent, the effect of each of these factors acting in opposition to each other is, respectively, 4 per cent and 1.5 per cent. Accordingly, the amount of oxygen passing into solution is only a small percentage less than that indicated by the theoretical formula. Furthermore, in the deduction of the theoretical formula no allowance has been made for the passage of nitrogen into solution.

The foregoing theoretical development for the amount of oxygen passing into solution during the ascent of an air bubble through a water column having an oxygen content of zero, is possible as a result of the adoption of the value h'. The determination of the value of this entity gives a clear insight into the degree of surface renewal during ascent of the bubble.

There is another line of theoretical reasoning which indicates more correctly what happens at the interfacial surface. This method does not, however, at once afford a clear insight into the degree of surface renewal.

At the bubble-to-water interfacial surface there is a concentration gradient which may be represented by

$$\frac{1-u}{\gamma}(c_{Ls}-c_L) = \chi\,(c_{Ls}-c_L),$$

where u $(c_{L_s} - c_L)$ is the concentration of oxygen at a distance γ from the interfacial surface and c_{L_s} the saturation value of oxygen in water, corresponding to the partial pressure of the oxygen in the air bubble. (The value represented by the relative concentration gradient χ will be discussed later.)

The quantity of diffused matter is proportional to the area of the surface through which diffusion takes place, to the concentration gradient and to the time, and is further determined by the value of the diffusion constant. The amount of oxygen passing into solution during ascent of the air bubble may therefore be formulated as:

$$Q = A\, k\, \chi\, (c_{L_s} - c_L)\, t$$

However, in the case considered, the diffusion of oxygen from air in water, this relation is not unreservedly correct. It only holds good when the same concentration gradient prevails both above and below the cross-section through which diffusion takes place. With a constant concentration c_{L_s} in the monomolecular boundary layer, this condition is not fulfilled.

$$Q = 2\, A\, k\, \chi\, (c_{L_s} - c_L)\, t \ldots . (5)$$

in full agreement with Stefan's formulae (3) (4). Accordingly the conditions for an interfacial surface at rest are expressed in the one case by: $Q = c_{L_s} \sqrt{\frac{kt}{\pi}}$ and in the other by $Q = 2\, c_{L_s} \sqrt{\frac{kt}{\pi}}$. For $c_L = 0$, the following can be deduced from Eq. 5:

$$Q = 2\, A\, k\, \chi\, \frac{V_{O_2}}{V_b}\, c_{atm}\, t$$

$$-\frac{dV_{O_2}}{V_{O_2}} = 2\, \frac{A}{V_b}\, \frac{k}{f}\, \chi\, c_{atm}\, dt$$

$$-\ln V_{O_2 h} = -\ln (1 - V_{dh}) = 2\, \frac{A}{V_b}\, \frac{k}{f}\, \chi\, c_{atm}\, t \ldots (6)$$

If the dimensions of the air bubble (A and V_b) are known and also the height of the water column h and the velocity of ascent v ($t = h/v$), the value of h' can be determined by means of Eq. 4 and the value of χ by means of Eq. 6, if V_{dh} has been found by experiment.

From Eqs. 4 and 6:

$$\chi = \frac{1}{\sqrt{\pi\, k\, t'}} \ldots . (7)$$

The value χ in Eq. 5 is in fact inversely proportional to the square root of the "time of existence of the hypothetical surface at rest" but of course does not show any proportionality to the time t.

Experimental Work*

The amount of oxygen that passes into solution from an air bubble during its ascent through a water column h, if the oxygen content of the water remains constantly equal to zero, can be determined by means of Eqs. 4 and 6, respectively, if the values of c_{atm}, f and k are known together with the dimensions of the air bubble (from which A/V_b is calculable), the velocity of ascent v and the value h' or χ as the case may be. These data are known or determinable, with the exception of h' or χ.

The oxygenation capacity, which can be determined for an aeration plant in practice or for an experimental setup in the laboratory, gives the velocity at which the oxygen drawn from the air supply becomes dissolved in the water, expressed in grams of oxygen per hour per cubic meter, assuming the oxygen content of the water to remain constantly equal to zero. If steps also are taken to determine the velocity at which the oxygen in the air is passed through the water, the ratio of the two velocities will give the percentage of oxygen that would become bound in the water if the oxygen content of the water had invariably been zero. In other words, this ratio will give the

* For additional information on the methods used in the experimental work, see reference (8).

same value V_{dh} as found from the Eqs. 4 and 6 respectively.

By equalizing the theoretical V_{dh} as found from Eqs. 4 or 6 with the experimental V_{dh}, the values for χ and h' can be determined.

If the values found for χ and h' at varying heights of the water column prove to be constant, it may be concluded that the formulae obtained by theoretical deduction are applicable. It may then be possible, from the absolute value of χ or h' from which t' is calculable, to draw certain conclusions regarding the progress of the diffusion process in the interfacial surface.

Air Bubbles

Air bubbles of different sizes were obtained with the aid of air diffusers of incrusted glass, Schott-Jena types G_1, G_2, G_3 and G_4, which have a pore size of 90–150μ, 40–90μ, 15–40μ and 3–15μ, respectively.

The average diameter of the air bubbles obtained with these air diffusers (calculated according to the volumetric percentage occupied) is dependent upon the rate of air supply. Therefore, the experiment was conducted in such a manner that the results of determination of the percentages of oxygen passing into solution from the air bubble at different heights of the water column could be compared with each other. This was achieved by using the same rate of air supply in making the determinations of the oxygenation capacity. Rates of air supply were approximately 20, 40, 60 and 80 ml. per min. The results obtained were plotted, and from these graphs the values for the oxygenation capacity were read at 20, 40, 60 and 80 ml. per min.

Bubble Dimensions

For the determination of the dimensions of the air bubbles the photographic method was employed. In order to determine the size and the

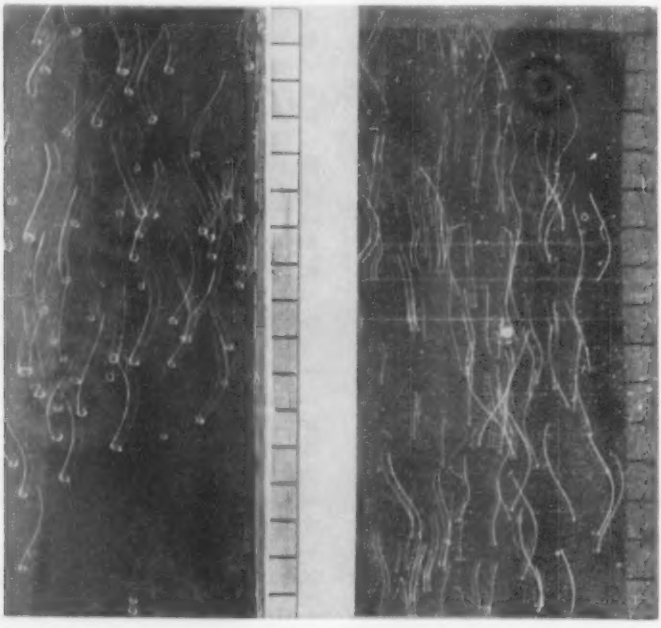

FIGURE 1.—Photographs for the determination of velocity of ascent of air bubbles.

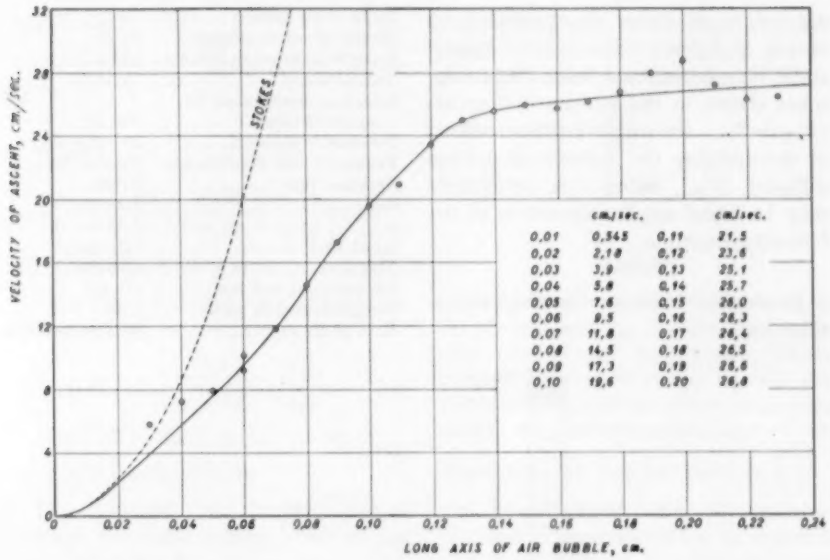

FIGURE 2.—Curve for the velocity of ascent of air bubbles of different sizes.

distribution of sizes, photographs of the air bubbles were taken with an exposure time of 1/5,000 sec. The bubble movement during the exposure time may therefore be ignored. Steps were taken to insure, as far as possible, that the rate of air supply at the time of taking these photographs was 20, 40, 60 and 80 ml. per min., respectively. The pictures obtained were enlarged 12.5 times by projecting on a screen, and the dimensions of the long and short axes of the air bubbles were measured.

For the purpose of calculation of the surface area and cubic contents of the air bubble it was assumed the bubbles have the form of an ellipsoid obtained by rotating the ellipse about its short axis.

Velocity of Ascent

For determination of the velocity of ascent photographs were taken with an exposure time of 1/10 or 1/25 sec., and at the commencement of this time interval an additional strong exposure was made with the electronic flash light during 1/5,000 sec. (Figure 1). A measurement was made of the vertical projection of the distance covered, and the velocity of ascent in centimeters per second was calculated. In order to permit this calculation the exposure times used must be accurately known.

Exposure times were determined by taking a series of photographs of a rotating disk or beam driven by a synchronous motor, using the exposure time to be determined. From the picture of the distance covered during the exposure time by a point on the circumference of the rotating disk and from the known linear peripheral velocity, an exact calculation of the exposure time could be made. The standard deviation for a series of seven photographs was 0.0004 sec. The results of the determinations of the velocity of ascent of nearly 1,000 air bubbles are shown in Figure 2.

Oxygen Solubility

The percentage by volume of oxygen that has passed into solution from the

air bubble in water deoxygenated by means of ferrous sulphate and caustic soda, was determined using the equipment shown in the schematic diagram (Figure 3). An insight into the method of determining the volume of oxygen diffused (V_{dh}), using this equipment may be had from consideration of the following example.

Example:—Data used in the calculations are:

Type of air diffuser	G_1
Height of water column	20 cm.
Volume of water in column	326 ml.
Temperature	19.6° C.
Injection needle used for constant flow	No. 20
Pressure in vessel A	204 mm. Hg
Pressure below air diffuser	29 mm. Hg
Aeration time	2 min.
c_o	1.36 mg./l.
c_t	5.46 mg./l.
c_{L_s} at 18.2° C.	9.01 mg./l.
Therefore, c_{L_s} at 19.6° C.	8.76 mg./l.
Air collected in 2 min.	172 ml.
Oxygen bound in water	1 ml.
Rate of air supply	86.5 ml. per min.

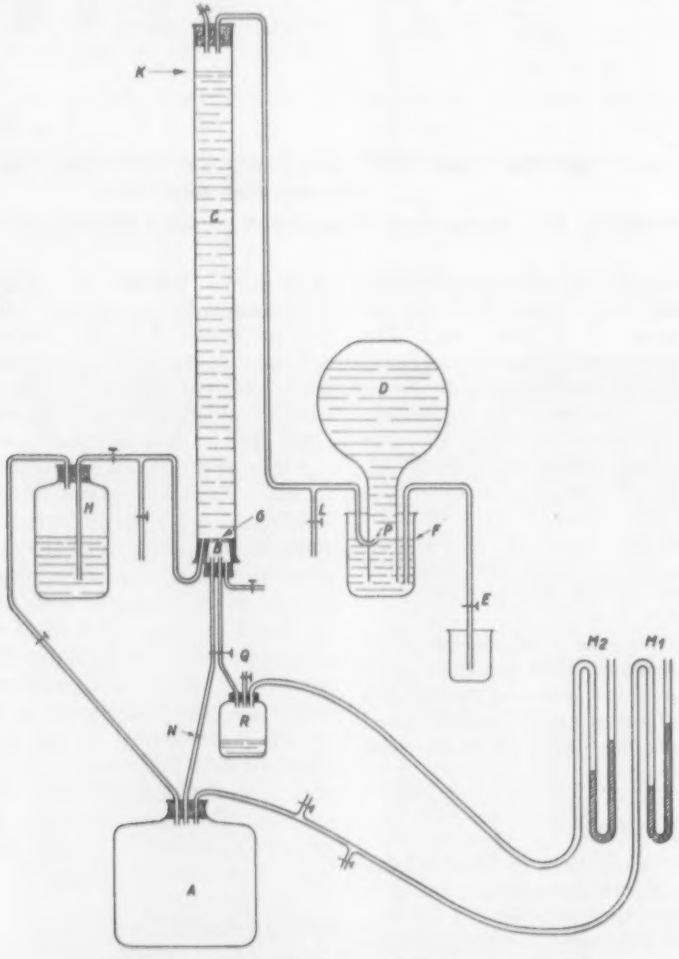

FIGURE 3.—Schematic diagram of the laboratory unit for oxygenation capacity determinations.

Calculation:

The equation is Oxygenation Capacity = O.C. at 10° C.

$$= 25.9 \frac{1}{t} \log \frac{c_{Ls} - c_o}{c_{Ls} - c_t} \sqrt{\frac{k_{10}}{k_t}}^{*}$$

$$= (25.9) \frac{60}{2} \log \frac{8.76 - 1.36}{8.76 - 5.46} 0.836$$

$$= 227.8 \text{ g./hr./cu. m.}$$

Then the calculation for Oxygenation Cap. at 20° C. is

$$= \text{O.C.}_{10° \text{ C.}} \frac{c_{Ls}\ 20° \text{ C.}}{c_{Ls}\ 10° \text{ C.}} \sqrt{\frac{k_{20}}{k_{10}}}$$

$$= 0.975 \times 227.8 \text{ g./hr./cu. m.}$$

$$= 222.1 \text{ g./hr./cu. m.}$$

The amount of oxygen passed through the water column in air during the experiment is

$$60 \times 86.5 \times 10^{-3} \times \frac{10^6}{326} \times 0.21 \times 1.429 \times \frac{273}{293}$$

= 4451 g. of oxygen per hour per cubic meter of water.

The deviation from 20° C. was in every case slight; accordingly, the temperature of the air throughout the experiments has been taken as 20° C.

$$\text{Hence } V_{da} = \frac{222.1}{4451}$$

$$= 0.0499$$

$$= 5.0 \text{ per cent.}$$

Results

Using the porous air diffusers G_1 to G_4, determinations of the V_{da} values were carried out with water column heights ranging from 20 to 300 cm. With the V_{da} values obtained and the results of the determinations of the dimensions of the air bubbles $\left(\frac{A}{V_b}\right)$ and the velocity of ascent (v), the values of χ and h' were calculated by means of Eqs. 4 and 6 or Eq. 7. The results are given in Tables I, II, III and IV.

These results show that during the ascent of the air bubble through a water column a considerable renewal of surface occurs. Speaking in terms of the concept used in the previous theoretical discussion, the interfacial surface at rest around the air bubble after every ½ to ⅓ cm. of travel is replaced by a new interfacial surface at rest. This means that when an air bubble has traversed a column height

* Values for $\sqrt{\frac{k_{10}}{k_t}}$ are available for temperatures of 9° C. to 24° C., see reference (5).

TABLE I.—Values of χ and h' Computed From Experimental V_{da} Values Estimated by Oxygenation Capacity Determinations; Air Diffuser G_1

Air (ml./min.)	20		40		60		80	
Average long axis (cm.)	0.154		0.186		0.184		0.178	
A/V_b (cm.$^{-1}$)	40.9		34.1		34.5		35.5	
v (cm./sec.)	26.1		26.6		26.5		26.5	
h (cm.)	χ†	h'‡	χ	h'	χ	h'	χ	h'
20	1,284	0.252	1,564	0.173	1,544	0.177	1,502	0.188
40	1,115	0.334	1,363	0.228	1,343	0.234	1,304	0.248
70	1,024	0.396	1,252	0.270	1,232	0.278	1,198	0.294
100	1,208	0.285	1,474	0.195	1,452	0.200	1,410	0.212
145	1,088	0.351	1,328	0.240	1,310	0.246	1,271	0.261
200	949	0.461	1,160	0.315	1,052	0.381	958	0.460
300	749	0.740	917	0.504	903	0.518	877	0.549

† χ in cm.$^{-1}$.
‡ h' in cm.

TABLE II.—Values of χ and h' Computed From Experimental V_{dh} Values Estimated by Oxygenation Capacity Determinations; Air Diffuser G_2

Air (ml./min.)	20		40		60		80	
Average long axis (cm.)	0.129		0.147		0.169		0.182	
A/V_h (cm.$^{-1}$)	48.8		42.8		36.9		34.8	
v (cm./sec.)	25.0		25.9		26.4		26.5	
h (cm.)	χ†	h'‡	χ	h'	χ	h'	χ	h'
20	1,280	0.243	1,514	0.180	1,791	0.131	1,907	0.116
40	1,059	0.355	1,250	0.264	1,480	0.192	1,575	0.170
70	1,228	0.264	1,190	0.291	1,370	0.224	1,427	0.207
100	1,495	0.179	1,336	0.231	1,422	0.208	1,431	0.206
145	1,100	0.329	1,163	0.305	1,321	0.241	1,376	0.223
200	1,039	0.369	957	0.449	1,028	0.398	1,048	0.384
300	933	0.457	893	0.517	946	0.470	929	0.489

† χ in cm.$^{-1}$.
‡ h' in cm.

TABLE III.—Values of χ and h' Computed From Experimental V_{dh} Values Estimated by Oxygenation Capacity Determinations; Air Diffuser G_3

Air (ml./min.)	20		40		60		80	
Average long axis (cm.)	0.076		0.084		0.101		0.118	
A/V_h (cm.$^{-1}$)	79.3		71.7		59.7		50.5	
v (cm./sec.)	13.4		15.7		19.8		23.2	
h (cm.)	χ†	h'‡	χ	h'	χ	h'	χ	h'
20	1,101	0.176	1,095	0.209	1,485	0.143	1,938	0.098
40	887	0.271	907	0.304	1,257	0.200	1,666	0.133
70	856	0.291	780	0.411	1,009	0.310	1,214	0.251
100	895	0.266	788	0.403	1,032	0.296	1,259	0.233
145	713	0.420	690	0.526	939	0.357	1,156	0.276
200	771	0.359	659	0.576	866	0.421	1,114	0.297
300	613	0.568	398	1.58	544	1.06	727	0.699

† χ in cm.$^{-1}$.
‡ h' in cm.

TABLE IV.—Values of χ and h' Computed From Experimental V_{dh} Values Estimated by Oxygenation Capacity Determinations; Air Diffuser G_4

Air (ml./min.)	20		40		60		80	
Average long axis (cm.)	0.060		0.057		0.074		0.083	
A/V_h (cm.$^{-1}$)	100		106		81.4		72.5	
v (cm./sec.)	9.5		8.9		12.9		15.3	
h (cm.)	χ†	h'‡	χ	h'	χ	h'	χ	h'
20	676	0.331	521	0.523	841	0.291	1,000	0.244
40	557	0.488	299	0.646	—	—	—	—
70	588	0.438	264	1.55	—	—	—	—
100	469	0.688	148	0.994	574	0.623	627	0.618
145	480	0.655	138	1.81	475	0.910	519	0.906
200	331	1.38	126	2.85	352	1.66	398	1.54
300	250	2.42	79	2.54	291	2.42	337	2.14

† χ in cm.$^{-1}$.
‡ h' in cm.

of three meters "a new interfacial surface at rest is formed a thousand times over."

In the case of smaller air bubbles the degree of renewal of the interfacial surface seems to be less. However, owing to the high value of $\frac{A}{V_b}$ the transfer of oxygen is naturally far more rapid than in the case of the larger air bubbles.

With air bubbles of approximately 0.08 cm. or more in diameter, the relative concentration gradient $\chi = \frac{1-u}{\gamma}$ has a value of about 1,300 cm.$^{-1}$.

The manner in which the concentration gradient at the "interfacial surface at rest" varies with time and what concentration gradient is represented by $\chi\ (c_{L_s} - c_L)$ will be discussed later. The value of χ is of primary concern and the corresponding value for h' can be determined with the use of Eq. 7.

Agreement of Experimental Results

After these preliminary observations on the intensity of renewal of the interfacial surface, a close study should be made of the results of the tables. Indeed, if the formulae found by theoretical deduction are strictly applicable, the values for χ and h' should be constant at given values for $\frac{A}{V_b}$ and v. Now this is not the case. The values for χ are greatest at a small height of the water column (20 cm.), after which they are more or less constant from 40 to 150 cm., and are definitely lower for a water column of 200 and 300 cm.

High Values in a Shallow Water Column

The conditions of aeration at the moment of introducing the air at the bottom of the water column are different from those in the rest of the column; there being a more rapid transfer of oxygen at the bottom. Moreover, additional aeration at the surface of the water column has more effect in the shallow water column than in higher water columns.

From this it follows that for a small height of the water column, the value found for V_{dh} may be expected to be somewhat higher than is indicated by Eqs. 4 and 6.

Low Values in a Deep Water Column

The lower values of V_{dh} and hence for χ in the case of water columns of 200 and 300 cm. demand more attention. For the two types of air diffuser G_1 and G_2, the average value of χ was calculated from the V_{dh} determinations at water column heights of 40, 70, 100 and 145 cm. for the various air supply rates (Table V).

TABLE V.—Average Values of χ

Air Rate (ml./min.)	χ (cm.$^{-1}$)	
	Diffuser G_1	Diffuser G_2
20	1,109	1,221
40	1,354	1,235
60	1,334	1,398
80	1,296	1,452

In the case of air diffuser G_2 it has been assumed that for an air supply rate of 20 ml. per min. χ = 879 cm.$^{-1}$, while in the case of air diffuser G_4 at this same rate χ = 550 cm.$^{-1}$ has been assumed.

With the aid of these average values for χ, curves were calculated for the percentage by volume of oxygen from the air bubble that has passed into solution after a height h (h up to 300 cm.) Curves for an air supply rate of 20 ml. per min. are shown in Figure 4. The results of the experimental determinations of the V_{dh} value are also shown. The graphs and experimental V_{dh} data for air supply rates of 40, 60 and 80 ml. per min. are omitted because they are similar.

It is evident that the value of V_{dh} as calculated from the determinations of the oxygenation capacity follows a

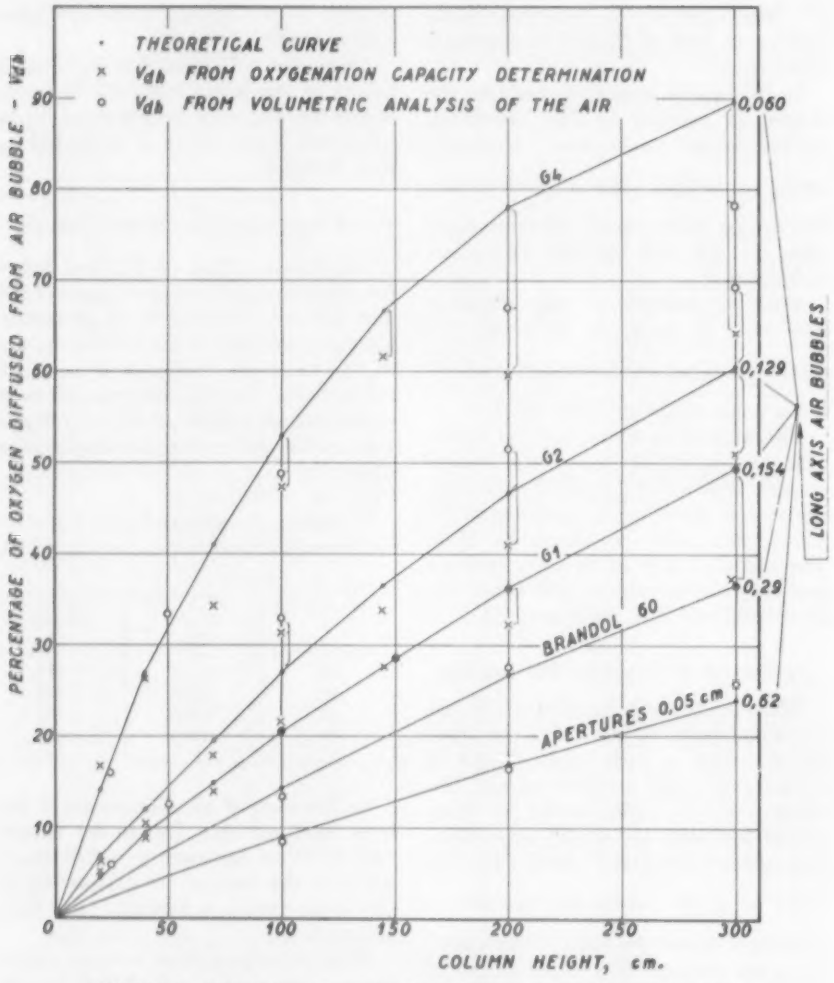

FIGURE 4.—Theoretical curves for the solution of oxygen from air bubbles and results of experimental investigations.

trend corresponding to Eqs. 4 and 6 up to a column height of about 150 cm. Above this height the values found are considerably lower. After considerable research the cause of the deviation from the theoretical line was found in all probability to be arithmetic.

For the saturation value c_{L_s} in this formula, the saturation value found after sufficiently long aeration in a water column of the respective height is used with any necessary correction for the difference in temperature. It is certain, however, that this saturation value is not the correct one. During ascent of the air bubble in water of low oxygen content, the partial tension of the oxygen in the air bubble decreases, as a result the "saturation value" of the interfacial surface also decreases. This is important for greater heights of the water column if the oxygen has dissolved to a considerable extent. The results of the oxygenation capacity

determinations may then be expected to be too low.

It is important both for the theory of oxygenation and for its practical application to obtain as clear an insight as possible into the validity of the formula found by theoretical deduction. Steps were taken to determine V_{da} directly by collecting the air bubbles after they have traversed a water column of a certain height and by carrying out a volumetric determination of the oxygen in the air.

Direct Volumetric Analysis of the Air

In carrying out this research the water column and the installations for water and air supply were used as shown in Figure 3. The upper part of the water column and the corresponding installation for determination of the oxygen content of the air collected are shown schematically in Figure 5.

After passage of the air through the water column (usually about 10 ml. in one-half minute) a certain quantity

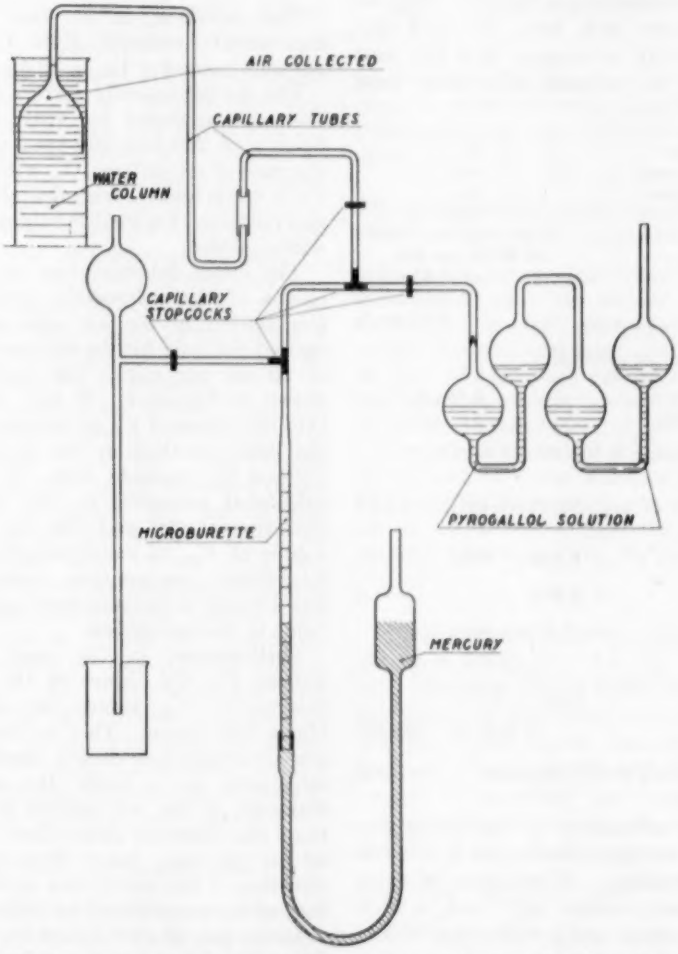

FIGURE 5.—Schematic diagram of the laboratory unit for the collection and analysis of air that has passed the water column.

of air was measured in the graduated burette. The oxygen was bound in the pyrogallol solution in the usual manner, after which the decrease in volume was measured. The method of calculating V_{dh} from this determination is shown in the following experimental example.

Example:—When making the calculation it is necessary to know the volumetric percentage of oxygen in the air. For this purpose use was made of the average of four determinations with the micrometric gas burette. The results were: 19.9, 20.0, 20.1 and 20.0 respectively, averaging 20.0 per cent oxygen by volume. The data used was:

Air diffuser	G_1
Water column	300 cm.
Temperature	18.1° C.
Water volume	4,600 ml.
Amount of air passed	11 ml. of air at a velocity of 20 ml. per min.
c_a	av. 1.35 mg./l.
c_t	av. 2.13 mg./l.
$c_{av.}$	1.74 mg./l.

Analysis of air collected:

Examined	6.45 ml.
Including N_2	5.63 ml.

Indicates $\frac{20}{80} \times 5.63 = 1.408$ ml. O_2

Amount of air absorbed in the water column

$$= 1.408 - 0.82$$

$$= 0.588$$

$$= 41.8 \text{ per cent}$$

$$\log (1 - V_{dh}) = \frac{9.43}{9.43 - 1.74} \times \log (1 - 0.418)$$

from which it follows that $V_{dh} = 48.5$ per cent.

This calculation is not altogether correct mathematically, but it is a fair approximation. A deviation of some importance occurs only with a high water column and a high value of V_{dh}. The result is then a small percentage too low.

The mathematically correct calculation is

$$(0.8+x)^2 \log \frac{0.8+x}{5x} = (0.8+\varphi)^2 \log \frac{c_{Ls} - \frac{c_L}{p}}{\frac{\varphi}{0.8+\varphi} c_{atm} - \frac{c_L}{p}} \quad . . (8)$$

in which $x = V_{O_2} = 1 - V_{dh}$ in the case $c_L = 0$ and $\varphi = V_{O_2}$ in the case $c_L = c_{av.} = \frac{c_o + c_t}{2}$.

This equation, in the case of the experimental example, gives $V_{dh} = 50$ per cent instead of $V_{dh} = 48.5$ per cent.

The mathematically correct calculation was employed for water column heights of 200 and 300 cm., while in the case of air diffuser G_4, with which V_{dh} is much higher, this calculation was also employed for smaller heights of the water column.

The direct determination of V_{dh} by means of the volumetric oxygen determination in the air collected was carried out only for the air supply rate of 20 ml. per min. The results are shown in Figure 4. It may be seen that the values of V_{dh} as determined by the direct method in the case of air diffuser G_1 coincide with the curve calculated according to Eq. 6. The deviation at 200 and 300 cm. in the values of V_{dh} as calculated from the established oxygenation capacity is due actually to the incorrect saturation value in the calculation.

Furthermore, in the case of air diffuser G_2, the values of the direct determined V_{dh} values are situated above the curve. This is due to a low air supply rate during these determinations, as a result the average diameter of the air bubbles was less than the diameter determined at the 20 ml. per min. rate. However, the direction of the curve does agree with that of the curve found by calculation.

In the case of air diffusers G_3 and G_4 the values found by direct determination differ somewhat from those of the

TABLE VI.—Contact Times Computed From Laminary Flow and From χ

Series	Av. Long Axis (cm.)	Av. Short Axis (cm.)	$\frac{1}{2}\pi d$ (cm.)	v (cm./sec.)	$\frac{\frac{1}{2}\pi d}{v}$ (sec.)	$t' = \frac{1}{\pi k \chi^2}$ (sec.)	χ (cm.$^{-1}$)
G_4 20 ml.	0.06	0.060	0.094	9.5	0.0099	0.0526	550
G_3 20 ml.	0.076	0.076	0.119	13.4	0.0089	0.0206	879
G_2 20 ml.	0.129	0.115	0.181	25.0	0.0072	0.0107	1,221
40 ml.	0.147	0.128	0.201	25.9	0.0078	0.0104	1,235
60 ml.	0.169	0.145	0.228	26.4	0.0086	0.0081	1,398
80 ml.	0.182	0.156	0.245	26.5	0.0092	0.0076	1,452
G_1 20 ml.	0.154	0.134	0.210	26.1	0.0080	0.0129	1,109
40 ml.	0.186	0.158	0.248	26.6	0.0093	0.0087	1,354
60 ml.	0.184	0.157	0.247	26.5	0.0093	0.0089	1,334
80 ml.	0.178	0.153	0.240	26.5	0.0091	0.0095	1,296
Brandol 60 Apertures 0.05	0.29	0.17	0.267	25.4	0.0105	0.0092	1,176
	0.62	0.31	0.487	25.0	0.0195	0.0092	1,318

curve established by calculation. This is possibly due to the fact that with these air diffusers the mixture of air bubbles is dissimilar with respect to the value χ. In the case of air diffuser G_1 the value of χ for the majority of the air bubbles is equal to 1,100 to 1,300 cm.$^{-1}$. For air diffusers G_3 and G_4 this value ranges from probably less than 300 to about 1,300 cm.$^{-1}$. Therefore, the calculation drawn up for a volumetrically determined average diameter may not be correct.

The research up to this point has been carried out with air bubbles up to a diameter of 0.2 cm. The bubbles used in practice in the activated sludge process are larger. By way of confirmation that the transfer of oxygen follows the trend of the theoretical curve, a further series of determinations was made with larger air bubbles, using a Brandol air diffuser and a porcelain plate with apertures of 0.05 cm. in diameter. The value of χ found with these larger air bubbles (average long axis 0.29 cm. and 0.62 cm., respectively) is approximately the same (χ = 1,176 and 1,318 cm.$^{-1}$) as with air bubbles of from 0.10 to 0.2 cm. in diameter.

Validity of the Theoretical Curve

Disregarding the comparatively small deviation in the case of the smallest air bubbles (diameter less than 0.1 cm.), the dissolution of oxygen from an ascending air bubble into water of low oxygen content follows exactly the same trend as the curve found by theoretical deduction.

The "Hypothetical Interfacial Surface at Rest"

From the mean value found for χ it is possible, with the aid of Eq. 7, to calculate a mean time of existence of the "hypothetical interfacial surface at rest" (t') for each series of experiments with a certain average diameter of the air bubbles.

If the water passing the air bubble is assumed to cover a distance equal to $\frac{1}{2}\pi d$, where d is the short axis of the air bubble, it follows that the time of contact of the water with the air bubble is $\frac{\frac{1}{2}\pi d}{v}$.

These values and the values for t' are shown in Table VI. For air bubbles with diameters of 1 to 3 mm. the times of contact as calculated in both ways are practically equal, amounting in both cases to approximately 0.01 sec. (mean values 0.0087 and 0.0095 sec.)

For larger air bubbles of about 0.6 cm. the t' calculated from χ is shorter than that calculated from $\frac{\frac{1}{2}\pi d}{v}$. For these larger air bubbles there are far more pronounced deformations during ascent, and the motion is also irregular.

Moreover, it is probable that with these bubbles the renewal of the interfacial surface is not equal at all parts of the surface. For very small air bubbles, t' is considerably greater than $\frac{\frac{1}{2}\pi d}{v}$. It is better not to attach any conclusions to this, but to await the result of determinations of χ with air bubbles of uniform size.

The close agreement, in the case of air bubbles of 1 to 3 mm. in diameter, between the "time of existence of the hypothetical interfacial surface at rest," t', and the time of contact as calculated from the measurement of the air bubble and the velocity of ascent, makes an important conclusion possible.

The fact is that this agreement is possible only if, during the flow of water past the air bubble, the boundary layers are actually at rest. This means the water must flow past the bubble without turbulence. If this condition were not fulfilled there would be a greater intake of oxygen due to the greater concentration gradient and hence t' would be shorter. This means that the hypothesis postulated at the outset, concerning the replacement of an interfacial surface at rest by another interfacial surface at rest after a time t' and a distance h' is found to agree with reality.

The only difference is in the hypothesis, the variation of the boundary layer is considered to be discontinuous and in reality the variation is a continuous one. This makes no difference in the result, for in both cases all the water flowing past the air bubble has passed through a time of contact with the air equal to t'.

The position is, therefore, that the air bubbles ascend through the water without a film of water molecules. In the case of air bubbles up to 3 mm. in diameter the water does not become turbulent as long as the air bubble is in contact with it. The amount of oxygen passing into solution during the time of contact can be calculated from Fick's law of diffusion (Stefan's formula).

It is possible to form an exact idea of the concentration gradient at the interfacial surface of the air bubble. This gradient follows the same trend as in an interfacial surface at rest with a time of contact of 0.009 sec. In Figure 6 the concentration gradient has

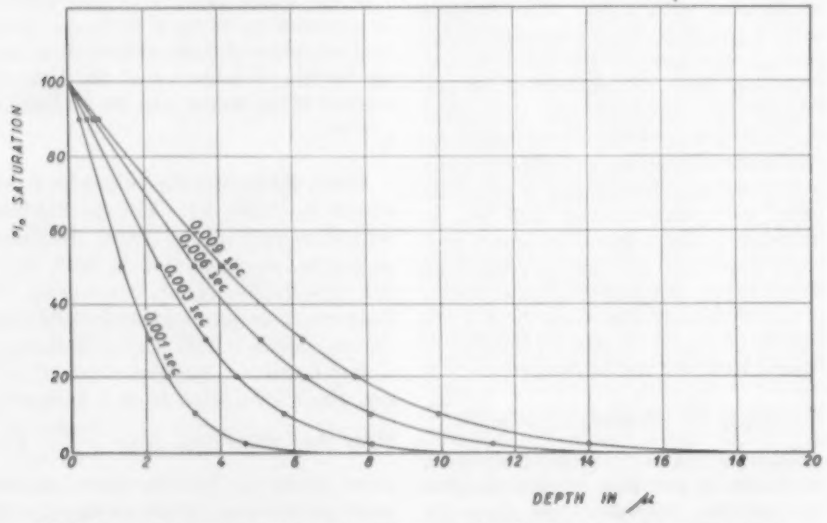

FIGURE 6.—The concentration gradient in the surface layers after short contact times.

been plotted for a contact time of 0.001, 0.003, 0.006 and 0.009 sec.

In the case of large air bubbles ascending in a narrow tube the water flows practically without turbulence past the air bubble and naturally has a much longer time of contact (6).

Relative Concentration Gradient

In view of the foregoing, further reference is made to Eqs. 5 and 7. What concentration gradient is represented by χ in these expressions?

In the case of $t' = 0.01$ sec., it is possible by means of Eq. 7, to deduce a value for χ of 1262 cm.$^{-1}$. Now the equation

$$c = c_{L_s}\left(1 - \frac{2}{\sqrt{\pi}}\int_o^y e^{-y^2}\,dy\right) \ldots (9)$$

in which $y = \dfrac{x}{2\sqrt{kt}}$, gives the concentration c at a depth x below the interfacial surface after a time t. The value of the expression

$$\left(1 - \frac{2}{\sqrt{\pi}}\int_o^y e^{-y^2}\,dy\right)$$

can be found from tables (7).

With the aid of these tables it may be calculated that in water at rest ($c_L = 0$) at the time $t = 0.01$ sec. the concentration 0.95 c_{L_s} is found at a depth below the surface $x = 0.3966\mu$. In the water layers directly adjacent to the boundary surface the concentration gradient at time $t = 0.01$ is then:

$$\frac{c_{L_s} - 0.95\,c_{L_s}}{0.3966 \times 10^{-5}} = 1{,}261 \text{ cm.}^{-1} \times c_{L_s} = \chi c_{L_s}.$$

This is the same value found above for χ by Eq. 7.

The relative concentration gradient represented by χ in Eqs. 5, 6, and 7 is found (in the case of air bubbles of 1 to 3 mm.) to be the relative concentration gradient occurring in water flowing without turbulence past the air bubble in the layers directly below the interfacial surface at the moment the contact with the air bubble has just ceased to exist.

Liquid Film Coefficient

A comparison between the equation

$$Q = 2\,A\,k\,\chi\,(c_{L_s} - c_L)\,t$$

and

$$Q = A\,K_L\,(c_{L_s} - c_L)\,t \ldots\ldots (1)$$

gives for the liquid film coefficient the value K_L, where

$$K_L = 2\,k\,\chi = 2\sqrt{\frac{kv}{\pi h'}} \ldots\ldots (10)$$

The Eq. 10 makes it possible in principle to calculate χ from the liquid film coefficient found by previous investigations. However, this will need to be done in a cautious manner, because, as has already been shown, the c_{L_s}, especially for small air bubbles, is a variable during ascent of the air bubble and cannot be determined or calculated in a simple manner. The values found for the liquid film coefficient K_L sometimes will be incorrect on this account.

Conclusions

1. The "liquid film theory" for the dissolving of gases in liquids gives an erroneous idea of what happens in the boundary layers. It leads to incorrect conclusions.

2. The "resistance" in the boundary is not caused by a "stagnant film," but follows from the fact that the rate of the diffusion process is inversely proportional to the square root of the time of contact.

3. Two equations there have been derived from which the quantity of oxygen, which goes into solution from an air bubble can be computed, if the bubble size and velocity of ascent are known and a magnitude, dependent on the rate of surface renewal, has been determined. The dissolving of the oxygen follows the theoretical curve.

4. When an air bubble ascends in

water, there is an intensive renewal of surface. A hypothetical "quiescent surface" is renewed after each $\frac{1}{4}$ or $\frac{1}{2}$ cm.

5. The hypothesis, in which the time of contact of the air bubble with the water is represented by a situation in which at certain time intervals the surface is suddenly replaced by a new surface which remains in perfect rest during the time interval, proves to agree with reality so that in the case of air bubbles from 1 to 3 mm. the water continually flows "in rest" without turbulence along the surface. There is no "stagnant film."

6. The concentration gradient $\chi\ (c_{L_s} - c_L)$ or in the case $c_L = 0$, $\chi\, c_{L_s}$ in Eq. 6 is the concentration gradient, which exists in the water layers at the boundary of the air bubbles at the moment the water leaves the air bubble, *i.e.*, at the moment the contact has just ceased to exist.

7. The relative concentration gradient χ in the case of air bubbles from 0.1 to 0.6 cm. has a value of about 1,300 cm.$^{-1}$. For air bubbles of 1 cm. a value of 1,000 cm.$^{-1}$ has been found. During the ascent of the air bubble χ is a constant.

8. In the dissolving of a poorly soluble gas (oxygen) in water the equation $Q = A\, k\, \frac{c_1 - c_2}{h}\, t$, where $\frac{c_1 - c_2}{h}$ is a concentration gradient for one particular moment, is not appropriate. It should be $Q = 2\, A\, k\, \frac{c_1 - c_2}{h}\, t$.

9. In the oxygenation capacity determinations in deep tanks, especially when small air bubbles are used, the equation

$$\text{O.C.} = 25.9\, \frac{1}{t} \log \frac{c_{L_s} - c_o}{c_{L_s} - c_t} \sqrt{\frac{k_{10}}{k_t}}$$

is unsuited, because the correct value of c_{L_s} cannot be determined or computed. In such cases a useful method gives the volumetric analysis of the air, with which can be determined the amount of oxygen that has gone into solution from the air.

10. The "liquid film coefficient" K_L of Lewis and Whitman (1) proved to be

$$K_L = 2\, k\, \chi = 2 \sqrt{\frac{kv}{\pi h'}}.$$

The values of K_L in literature sometimes may be incorrect because of incorrectness of c_{L_s}.

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TRANSFORMATION AND EFFECTS OF REDUCED SULFUR COMPOUNDS IN SLUDGE DIGESTION *

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Many studies have been done to determine if a particular waste can be treated by digestion. In most cases the studies consist of taking a sample of the waste, mixing it with various amounts of sewage sludge, and determining the degree of digestion.

Less study has been done to determine the effect of a particular compound upon digestion. Information of this nature would be desirable for preparing a list of substances which interfere with digestion, together with the approximate concentration at which interference begins, and the methods used to overcome the interference. One would only need then to analyze the particular waste in question for these substances in order to obtain an idea of the feasibility of digestion of the waste.

Of all the sulfur compounds present in sewage and industrial wastes, sulfide has received the most attention for various reasons. Sulfide is a common form of sulfur in many wastes which receive treatment by digestion. Oxidized forms of sulfur are reduced to sulfide during digestion. Moreover, sulfide has definitely been shown to interfere with the digestion process.

Some of the first experiments pertaining to the effects on digestion of the reduced sulfur compounds in sewage sludge were made without the knowledge that these effects were caused by the reduced sulfur compounds present. Favre (1) showed that stagnant effluents retarded digestion, and Guth and Feigle (2) noted that enzymes were greatly reduced when the sewage became septic. Heukelekian (3) observed that total gas yields were reduced with increasing periods of septicization.

The retardation of digestion by septic sewage was soon correlated to the presence of sulfides. Rudolfs and Baumgartner (4) found that thiosulfates produced sulfides at a greater rate than sulfites and sulfates. In working with several inorganic sulfur compounds, Lassen (5) showed that generally the sulfur compounds had a tendency to increase liquefaction, but retarded mineralization and gasification. Similar conclusions were reached from experiments on the digestion of strawboard wastes and the fermentation of sodium and calcium acetate by Noordam-Goedewagen, Manten and Muller (6).

Studies of the digestion of white water by Rudolfs and Amberg (7) showed that the reduction of sulfate to sulfide resulted in interference with digestion. The presence of 200 p.p.m. sulfide added as sodium sulfide almost completely inhibited the digestion of the white water, whereas the same amount of sulfide had a greater effect on the fermentation of sodium acetate substrates. The differential effect of sulfide on acid production and methane fermentation results in an accumulation of volatile acids.

This paper is concerned with some of the transformations and effects of sulfur compounds upon sewage sludge digestion.

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Method and Procedures

Several series of experiments were run to study (a) the transformations and effects of various concentrations of sulfite; (b) the effects of various concentrations of sulfide; (c) the effects of various sulfide salts; and (d) the effects of cystine and methionine. For each series, the results were compared with a control containing no added sulfur compound.

The sludge used consisted as nearly as possible of equal amounts of volatile solids in the fresh and the ripe (seed) sludge. A 1.5 l. portion of the sludges was placed in a bottle connected to a gasometer to measure the volume of gas produced. The gas was collected over water saturated with sodium chloride so that qualitative and quantitative analyses of the gas could be made.

Digestions were continued for at least 56 days, during which periodic analyses were made for solids, ash, pH, volatile acids and total sulfide in the sludge. In the experiment to determine the transformation of added sulfite, the analyses included sulfate, sulfite, thiosulfate and total sulfur. Organic sulfur was calculated as the difference between total sulfur and the sum of the inorganic sulfur analyses. The volume of gas produced was measured and tested periodically for oxygen, carbon dioxide, combustibles and hydrogen sulfide.

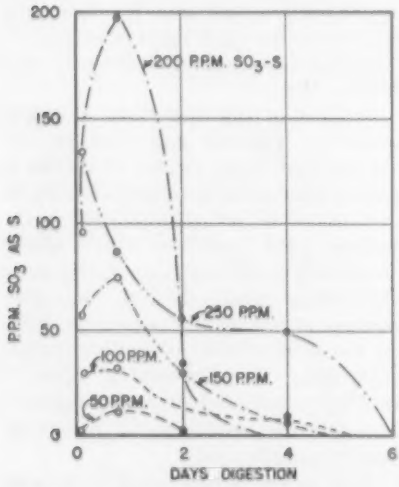

FIGURE 1.—Changes in sulfite content with the addition of various concentrations of sulfite to a digestion mixture of fresh solids and ripe sludge.

All the digestions were performed at 20° C., and the volumes of gas recorded were collected at this temperature and not corrected for standard temperature and pressure. Except where specified in the experiment with the various sulfides, all sulfur compounds are reported on the basis of sulfur. The sampling bottles were thoroughly shaken before samples were removed, and the bottles connected to the gasometers were shaken before gas measurements were made.

Transformation and Effects of Sulfite

The sodium sulfite additions chosen for this experiment were 50, 100, 150, 200 and 250 p.p.m. on the basis of sulfur.

The transformation of the sulfite is revealed best by Figures 1, 2 and 3, which show the sulfite, thiosulfate, and sulfide contents, respectively, of the sludges during digestion. Figure 1 indicates that the initial recoveries of sulfite were low. Also, the greater the initial sulfite concentration, the slower it disappeared. All the sulfite disappeared by the seventh day. Sulfite was not found in the control at any time, although a small amount of thiosulfate was always present (Figure 2). Thiosulfate increased for the first one to three days and then decreased to about the same value as in the control within about 10 days for all sulfite concentrations. The amount of thiosulfate present and the length of time it remains are directly proportional to the initial sulfite concentration.

The sulfide concentration shown in Figure 3 reveals that there was a continual increase in the sulfide content

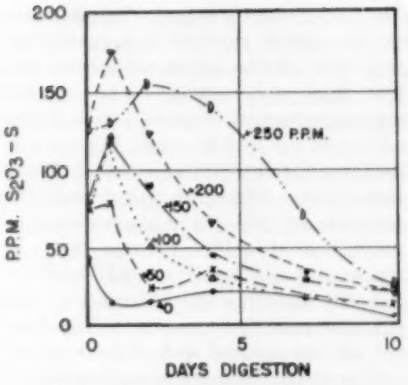

FIGURE 2.—Changes in thiosulfate content with the addition of various concentrations of sulfite to a digestion mixture of fresh solids and ripe sludge.

of the control, whereas the sulfide content with the added sulfite increased during the early part of the digestion, but then reached a constant level. The increase in sulfide with the addition of even 250 p.p.m. sulfite was less than 100 p.p.m. greater than the control at the end of the experiment.

Figure 4 compares the various forms of sulfur compounds obtained with the addition of 250 p.p.m. of sulfite. As the sulfite disappears, the thiosulfate increases; then, with continuing digestion, the thiosulfate disappears with a corresponding increase in the sulfide. This indicates that thiosulfate is a determinable intermediate in the reduction of sulfite to sulfide during sewage sludge digestion.

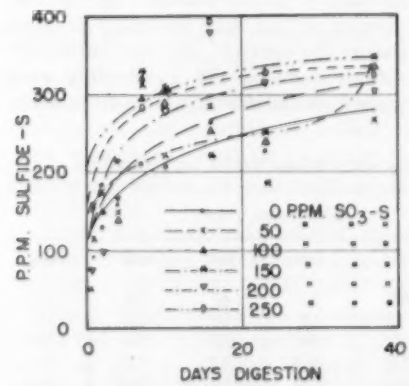

FIGURE 3.—Changes in sulfide content with the addition of various concentrations of sulfite to a digestion mixture of fresh solids and ripe sludge.

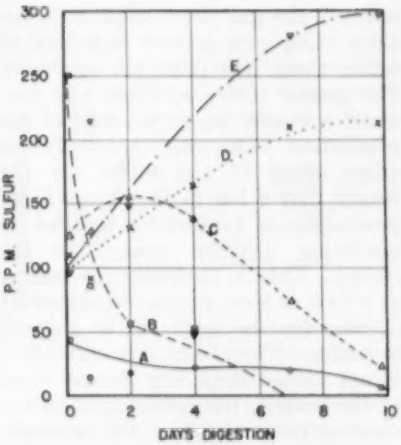

FIGURE 4.—Changes in the various forms of sulfur compounds with the addition of 250 p.p.m. of sulfite. (A) thiosulfate in control; (B) sulfite with sulfite addition; (C) thiosulfate with sulfite addition; (D) sulfide in control; (E) sulfide with sulfite addition.

At no time during the experiment was there any significant amount of sulfate present in any of the samples.

The effects of the added sulfite on sludge digestion are shown by the cumulative gas production in Figure 5. The addition of only 50 p.p.m. sulfite

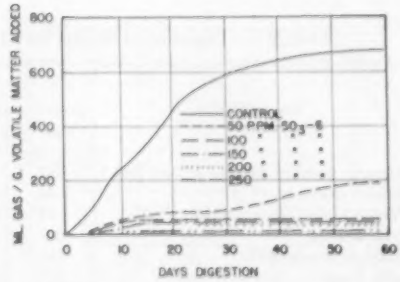

FIGURE 5.—Effect of the addition of various concentrations of sulfite on gas production.

reduced the gas production considerably, while with greater additions of sulfite there was little gas produced. The greater sulfite additions also produced a greater lag in the start of gas production. This was apparently the direct effect of the sulfite for the reason that a lag in the start of gas production is normally accredited to something initially present in the sludge. This is obviously the sulfite, as it took at least a measurable amount of time for the sulfite to be reduced to sulfide. Therefore, since the sulfide is not immediately any greater than in the control, the sulfite must be the cause of the initial lag. Furthermore, in experiments with the addition of sulfide, a sulfide concentration similar to the sulfite concentration which produced a lag in gasification resulted in no lag. In other words, if all the sulfite added were immediately reduced to sulfide, there would have been less interference with gasification.

Analyses of the gas produced showed the greater sulfite additions reduced the ratio of combustibles to carbon dioxide. Never was there as much as 1 p.p.m. sulfide found in any of the gas collected.

The effects of the sulfite additions upon digestion are summarized in Table I. The addition of only 50 p.p.m. sulfite reduced the volume of gas per gram of volatile matter added by more than one-third. Greater additions of sulfite resulted in the production of even less gas. The decrease in the solids content was about the same for all the sulfite additions; but less than the control. The volatile matter reduction decreased with sulfite additions up to 100 p.p.m. but greater sulfite additions did not result in lower reductions. The high value with the addition of 200 p.p.m. is anomalous. Additions of sulfite greater than 50 p.p.m. did not lower the pH further, but did slow up the lowering of the pH and retarded recovery. The final pH of the control and the 50 p.p.m. sulfite sample showed complete recovery. With the addition of 200 and 250 p.p.m. the pH reached its lowest value on the final day of the experiment. Only a small amount of volatile acids remained in the control and the 50 p.p.m. sulfite sample at the end of the experiment. Considerably more remained with the higher sulfite additions.

The results show that sulfite is reduced in a digester, first to thiosulfate and then to sulfide. With the addition of 250 p.p.m. sulfite, all the sulfite had disappeared by the sixth day. The thiosulfate produced was, in turn, all reduced to sulfide by the 10th day.

The presence of even 50 p.p.m. sulfite greatly retards gasification. This amount of sulfite resulted in the production of less than one-third as much gas as the control. Greater sulfite additions reduced the gasification even more. With the greater sulfite addi-

TABLE I.—Digestion of Sludge for 73 Days with Various Sulfite Concentrations

Determination	SO_3-S Added (p.p.m.)					
	0[1]	50	100	150	200	250
Gas/g. volatile matter added (ml.)	681	208	79	48	45	13
Solids decrease (%)	16	12	12	10	13	11
Volatile matter reduction (%)	31	21	10	12	26	12
pH, lowest	6.3	5.6	5.5	5.6	5.7	5.8
Day of lowest pH	7th	14th	16th	23rd-37th	72nd	72nd
pH, final	7.3	7	6.2	5.8	5.7	5.8
Final volatile acids (meq./l.)	0.2	6	29.6	26.5	27.9	24

[1] Control.

tions there was also a lag in the start of gasification. This indicates it is the sulfite itself that interferes with gasification, since at the beginning of digestion, the sulfite was not reduced to thiosulfate and sulfide.

The presence of sulfite has some effect on the solids reduction; however, it has a greater effect on the volatile matter reduction.

The lack of gasification and the rapid drop in pH with the presence of 50 p.p.m. sulfite indicate volatile acids were being produced, but not utilized. Although volatile acids were not determined throughout the experiment, the lowering of the pH is a good indication of the volatile acids content. Therefore, 50 p.p.m. sulfite interferes only with gasification. However, with sulfite additions greater than 100 p.p.m. the pH was lowered more slowly, indicating volatile acids were produced more slowly. Since the acids were not utilized (there was no gasification), they tended to build up gradually as indicated by the slowly decreasing pH throughout the experiment. The slightly lower final volatile acids concentration with the addition of 250 p.p.m. sulfite also shows there was less volatile acids production at this sulfite concentration.

Effects of Various Concentrations of Soluble Sulfide

Since it was shown that sulfite and thiosulfate are reduced to sulfide, it was considered desirable to study the effects of sulfide upon sewage sludge digestion.

Fifty, 100, 150, 200 and 250 p.p.m. sulfur as sodium sulfide was added to mixtures of fresh solids and ripe sludge. The results were compared with the same sludge mixture containing no added sulfide. Because this compound is highly alkaline, it was first dissolved in a minimum of water and then neutralized with 1:1 hydrochloric acid. The total sulfur in the

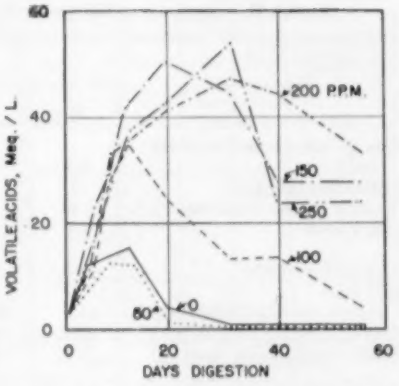

FIGURE 6.—Volatile acid production in digesting sludge mixtures with the addition of various concentrations of sulfide.

sludge was determined, and the increase in the total sulfur was taken as an indication of the actual sulfide addition. By this method of calculation the actual sulfide additions were, respectively, 45, 56, 102, 127 and 80 p.p.m. Whereas, these values are not absolutely accurate, they are considered to be more correct. Nevertheless, the theoretical values for the additions are used for all the figures and tables.

The production of volatile acids with the various sulfide additions is shown in Figure 6. There was little difference between the control and the addition of 50 p.p.m. sulfide sample. With greater additions of sulfide, the rate of volatile acids production was

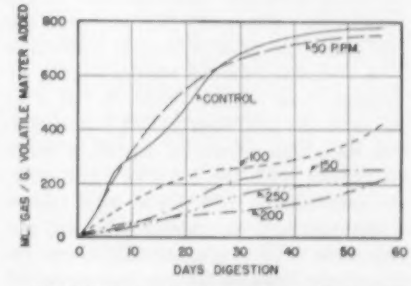

FIGURE 7.—Effect of various concentrations of sulfide additions on gas production.

TABLE II.—Digestion of Sludge for 56 Days with Various Sulfide Concentrations

Determination	Sulfide Added (p.p.m.)					
	0[1]	50	100	150	200	250
Gas/g. volatile matter added (ml.)	786	756	426	254	192	213
Solids decrease (%)	24	22	19	18	18	25
Volatile matter reduction (%)	46	48	33	30	27	29
pH, lowest	6.0	6.1	5.6	5.4	5.5	5.6
Day of lowest pH	5th	5th	12th	12th	19th and 28th to 35th	12th
pH, final	7.2	7.4	7.1	6.8	6.7	7.1
Greatest volatile acids (meq./l.)	14.9	12.7	35.2	50.5	47.1	53.9
Day of greatest volatile acids	12th	8th	12th	19th	26th	26th
Final volatile acids (meq./l.)	0.12	0.12	3.95	28.0	33.1	24.5

[1] Control

generally higher. After a maximum was reached the volatile acids decreased slowly.

Figure 7 shows the effects of the sulfide upon the cumulative gas production. Again, there was no significant difference between the control and the addition of 50 p.p.m. sulfide. However, the addition of 100 p.p.m. reduced the gas production to one-half that of the control. Greater sulfide additions resulted in the production of even less gas, but the addition of even 250 p.p.m. did not completely inhibit gasification.

Analyses of the gas produced showed no significant amount of hydrogen sulfide present at any time. During digestion, the ratio of combustibles to carbon dioxide continually increased, showing methane was produced despite the small amount of gas produced. However, this increase was less with the greater sulfide additions.

A summary of the results of the digestion is shown in Table II. It may be seen the addition of 50 p.p.m. sulfide had little effect on digestion when compared with the control. However, the addition of 100 p.p.m. and higher concentrations of sulfide affected the digestion. The greater additions reduced the volume of gas per gram of volatile matter added. The solids were not decreased as much as in the control, and the volatile matter reduction was considerably less than in the control. With all sulfide additions greater than 50 p.p.m., the pH reached about the same low value, the lowest being 5.4. However, the greater the sulfide addition, the later the lowest pH occurred and the slower was the recovery. Similarly, the greater the sulfide additions the greater the volatile acids concentration, which was reached later and recovered slower.

The results of this experiment show the limit of neutral soluble sulfide that can be tolerated in sewage sludge digestion is between 50 and 100 p.p.m. The sulfide appears to interfere with digestion by inhibiting utilization of volatile acids which accumulate and are not gasified. Apparently there is a slow utilization of volatile acids as they slowly disappeared on continuing digestion. There appears to be no interference with volatile acid production in the range of sulfide studied. The sulfide decreases the liquefaction of the solids to a small extent, and interferes to a greater extent with the reduction of volatile matter.

Effects of Various Sulfides

The previous experiment to determine the effects of various concentrations of sulfide was performed with

neutral soluble sulfide as sodium sulfide. Since sulfide may be present in many other forms in a digester, different sulfides were added in their normal state without neutralization.

The sulfides used were hydrogen sulfide, ferric sulfide, ferrous sulfide, calcium sulfide, ammonium sulfide and sodium sulfide. The results were compared with a control containing no added sulfide. The concentration of sulfide to be added was based on the amount of neutralized sodium sulfide shown to interfere markedly with the digestion. On this basis, a sulfide concentration of about 150 p.p.m. was used, which is roughly equivalent to 400 mg. per l. of sodium sulfide. Therefore, a total of 400 p.p.m. of each of the sulfide salts was added to the sludge for digestion.

The hydrogen sulfide added to the mixture of the sludges was prepared by bubbling the gas through distilled water and then standardizing the solution. The actual sulfide concentration added to the sludge was 377 p.p.m.

The ferric sulfide was prepared by mixing equivalent amounts of ferric chloride and sodium sulfide; keeping the resulting colloidal sol under alkaline conditions. This sol was standardized in the wet state, and resulted in an actual sulfide addition to the sludge of 185 p.p.m.

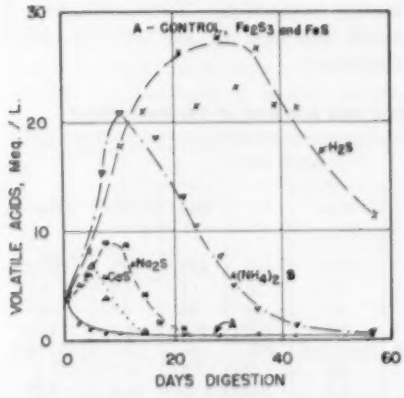

FIGURE 8.—Volatile acid production in digesting sludge mixtures with the addition of different salts of sulfide.

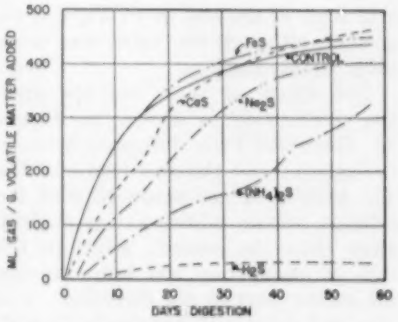

FIGURE 9.—Effect of different salts of sulfide on gas production.

Both the ferrous and the calcium sulfides were added as the anhydrous powder. Their respective sulfide additions to the sludge were 146 and 178 p.p.m.

The ammonium sulfide was added as a liquid, and the sodium sulfide was added as the hexahydrate; producing sulfide additions to the sludge of 188 and 164 p.p.m., respectively.

The effects of the various sulfides on the volatile acids production are shown in Figure 8. There was no difference between the control, and the ferric and the ferrous sulfides. For these three, the highest volatile acids value occurred in the initial mixture, and the concentration dropped rapidly to a negligible value. The calcium sulfide showed a slight increase in volatile acids, but these dropped to the level of the control by the 15th day. The sodium sulfide produced the next highest maximum, but this was considerably less than the amount formed in the experiment in which the sodium sulfide was neutralized. Within 22 days the concentration also reached that of the control. The ammonium sulfide reached a still higher concentration of volatile acids, but this returned to the value of the control by the 45th day. The greatest concentration was produced with the hydrogen sulfide, and this was

still high at the end of 57 days of digestion, although the value was dropping at this time.

The effect of these sulfides upon the gasification is shown in Figure 9. There was little difference between the control and the two iron sulfides. The amount of gas produced with the calcium sulfide addition was slightly more than the control, although the rate of gasification was lower during the earlier part of the digestion. The rate and volume of gas produced with the addition of sodium sulfide was lower than the control. Again, gasification was higher than was noted with the experiment in which the sodium sulfide was neutralized. The addition of the ammonium sulfide produced an even greater lag and less total gas. However, from the rate of gasification at the end of the experiment, it appears that the total amount of gas would have reached that of the control had the experiment been continued longer. Very little gas was produced with the addition of hydrogen sulfide.

Analyses of the gas produced showed at no time was there more than a trace of hydrogen sulfide, even with the addition of hydrogen sulfide. The ratio of combustibles to carbon dioxide was similar to the control in all cases, and increased with continued digestion.

The results of the experiment are summarized in Table III. The hydrogen sulfide decreased the volume of gas per gram of volatile matter added to very low values. There was less mineralization and considerably less volatile matter reduction than in the control. The pH was the lowest, and the volatile acids concentration the highest of all the sulfide additions. The recovery was also less than the others.

The volume of gas produced with the ferric sulfide addition was comparable to that of the control. However, the solids decrease and volatile matter reduction were less than that of the control. There was no effect on the pH and the volatile acids.

The only effect the ferrous sulfide had on digestion was a slight reduction in the amount of liquefaction.

The calcium sulfide produced slightly more gas than the control. However, this addition lessened the solids and volatile matter reduction. There was also a slight increase in volatile acids.

The addition of ammonium sulfide lessened gas production somewhat, but there was almost no decrease in solids. Volatile matter reduction was also the lowest of all the sulfide additions. There was a marked increase in the volatile acids concentration with a subsequent lowering of the pH.

The sodium sulfide generally slowed digestion as noted by the lower gas production, solids decrease and volatile matter reduction. There was also a small increase in the volatile acids content.

TABLE III.—Digestion of Sludge for 57 Days with Additions of Various Sulfides

Determination	Sulfide Added						
	Control	H_2S	Fe_2S_3	FeS	CaS	$(NH_4)_2S$	Na_2S
Gas/g. volatile matter added (ml.)	434	30	436	448	453	321	402
Solids decrease (%)	19	12	12	15	11	3	10
Volatile matter reduction (%)	45	25	33	44	38	18	34
pH, lowest	6.3	5.6	6.5	6.3	6.4	6.1	6.5
Day of lowest pH	4th	10th–14th	2nd–10th	4th	4th	10th	4th–10th
pH, final	7	6.6	6.9	7	7	7	7
Greatest volatile acids (meq./l.)	3.4	27.8	3.4	3.6	7	20.6	8.7
Day of greatest volatile acids	1st	28th	1st	1st	4th	10th	7th–10th
Final volatile acids (meq./l.)	0.4	11.2	0.4	0.4	0.3	0.8	0.3

The overall results of the experiment show two factors which govern the effects of sulfides on sewage sludge digestion: (a) the solubility of the sulfide; and (b) the concentration of the soluble sulfide. The ferric, ferrous and calcium sulfides are insoluble, and had little effect on digestion. The sodium, ammonium, and hydrogen sulfides are completely soluble in their respective concentrations of 164, 188 and 377 p.p.m. as used in the sludge. This is also the order of increasing interference with digestion. Apparently the limiting concentration of sulfide to permit good digestion is in the range of 200 p.p.m. of soluble sulfide.

This concentration does not appear to agree, at first, with the value previously determined with the addition of neutralized sodium sulfide. The difference may be attributed to the lack of neutralization in the present experiment. Both the sodium and the ammonium sulfides are extremely alkaline. This alkalinity, when not first neutralized, may serve to improve the digestion by neutralizing the volatile acids produced, thereby lessening their subsequent lowering of the pH.

Effects of Cystine and Methionine

Cystine and methionine were chosen for study as representative of all sulfur containing organic compounds that may be present in sewage. These compounds are products of human metabolism. However, their concentrations in sewage are quite low. In order to have sufficient sulfur present to study, these compounds were added to sewage sludge in concentrations of 100, 500, and 1,000 p.p.m. on the basis of their sulfur contents, and the digestions compared with a control containing no added sulfur compound.

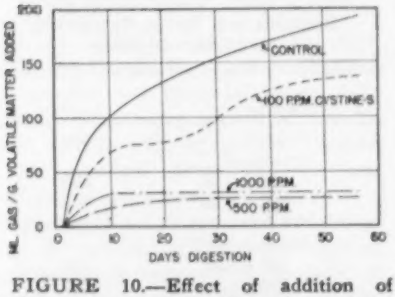

FIGURE 10.—Effect of addition of various concentrations of cystine on gas production.

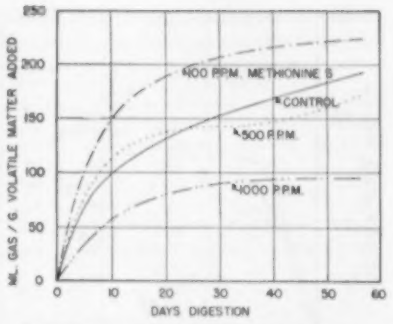

FIGURE 11.—Effect of addition of various concentrations of methionine on gas production.

The effects of the cystine on gasification are shown in Figure 10. It may be seen that even the 100 p.p.m. addition lowered the gas production somewhat, whereas the addition of 500 and 1,000 p.p.m. resulted in low gasification.

On the other hand, the effects of the methionine on gasification as shown in Figure 11 reveal the addition of 100 p.p.m. actually stimulated the digestion. The addition of 500 p.p.m. of methionine did not materially affect gas production. However, gas production was lowered with the 1,000 p.p.m. addition.

Analyses of the gas produced showed that with the exception of the 100 p.p.m. methionine addition, the ratio of combustible gas to carbon dioxide was decreased with the addition of organic sulfur. Sulfide was not found in the gas from the control, in the 100 p.p.m. cystine addition, and in the 100 and 500 p.p.m. methionine additions. So little gas was produced with the 500 and 1,000 p.p.m. cystine additions that

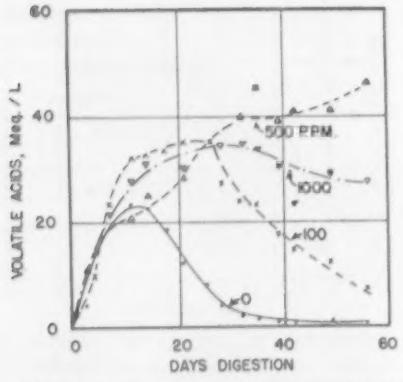

FIGURE 12.—Volatile acid production in digesting sludge mixtures with the addition of various concentrations of cystine.

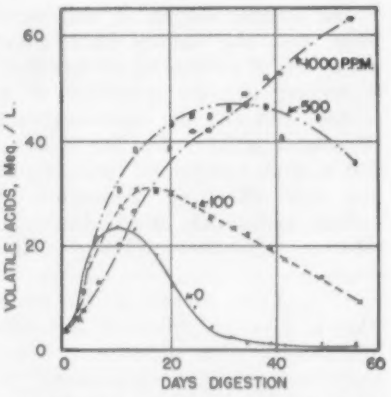

FIGURE 13.—Volatile acid production in digesting sludge mixtures with the addition of various concentrations of methionine.

only one determination for sulfides could be made for each: 639 and 2,992 p.p.m., respectively. With the 1,000 p.p.m. methionine addition, 498 p.p.m. sulfide were found on the 11th day, 334 p.p.m. on the 22nd day, and none on the 56th day.

The volatile acids concentrations during digestion are shown in Figure 12. The increase in volatile acids in the control reveals that digestion in the control was not as satisfactory as in the other experiments. However, the addition of 100 p.p.m. cystine resulted in an even greater volatile acids concentration, which decreased near the end of the experiment to the almost negligible amount found in the control. The volatile acids concentration with the 500 and 1,000 p.p.m. additions were slightly greater than with the 100 p.p.m. addition, and did not decrease toward the end of the experiment.

Figure 13 shows the volatile acids with the methionine additions. The 100 p.p.m. addition resulted in a higher concentration than in the control, but this dropped steadily toward the end of digestion. With the 500 p.p.m. addition, the volatile acids increased to an even greater value and showed only a slight indication of dropping at the end of the experiment. The 1,000 p.p.m. addition resulted in the greatest concentration of volatile acids of all the organic sulfur additions, and the acids were still increasing at the end of the experiment.

The degree to which the organic sulfur compounds broke down into sulfides during the digestion is shown by the sulfide concentrations summarized in Table IV. Considerable amounts of sulfide were produced with the cystine additions, and only a small amount was produced with the methionine. To this must be added the sulfide found in the gas, showing that a great amount of sulfide was produced

TABLE IV.—Approximate Sulfide Concentrations in Sludge During Sewage Sludge Digestion with Various Cystine and Methionine Additions

Digester	Sulfide (p.p.m.)
Control	300
Cystine-S	
100 p.p.m.	400
500 p.p.m.	450
1,000 p.p.m.	750
Methionine-S	
100 p.p.m.	325
500 p.p.m.	400
1,000 p.p.m.	400

TABLE V.—Digestion of Sludge for 56 Days with Various Cystine and Methionine Additions

Determination	Control	Cystine-S p.p.m.			Methionine-S p.p.m.		
		100	500	1,000	100	500	1,000
Gas/g. volatile matter added (ml.)	192	134	26	30	223	170	94
Solids decrease (%)	23	23	18	13	23	17	19
Volatile matter reduction (%)	59	53	38	28	55	44	39
pH, lowest	5.9	5.4	5.6	5.6	5.6	5.7	5.5
Day of lowest pH	11th	14th	56th	11th	11th–14th	14th & 28th	56th
pH, final	7	7	5.6	6.5	6.9	6.1	5.5
Greatest volatile acids (meq./l.)	23.2	36	46.8	35.7	31	46.6	64.6
Day of greatest volatile acids	11th	25th	56th	32nd	21st	35th	49th
Final volatile acids (meq./l.)	0.5	7.2	46.8	27.4	8.5	41.9	61.9

from the cystine and a smaller amount from the methionine.

Table V presents a summary of the experiment with organic sulfur compounds. The addition of 100 p.p.m. cystine reduced gasification, and the 500 and 1,000 p.p.m. additions almost completely inhibited gasification. More gas per gram of volatile matter added was produced with the addition of 100 p.p.m. methionine than with the control. Gas production was slightly less than the control with the 500 p.p.m. methionine addition, and was only one-half the control with the 1,000 p.p.m. addition. The addition of 100 p.p.m. of both the organic sulfur compounds had no effect upon the solids reduction, but greater cystine additions resulted in less reduction than similar methionine additions. The volatile matter reduction was similar to the solids reduction. A low pH value was reached in all of the experiments; however, it was constantly lower and showed less recovery with the methionine additions, despite the fact that gasification was greater with these additions. The volatile acids were generally the inverse of the pH, again the greatest values resulted from the greatest methionine additions.

Cystine is soluble to the extent of only 27 p.p.m. at 20° C., whereas methionine is completely soluble at all the concentrations used. Therefore, any effects of the cystine upon digestion must be attributed to the breakdown of the cystine into compounds which interfere with digestion. It is known (8) that one of the anaerobic decomposition products of cystine is sulfide, whereas sulfide is not a normal product of the anaerobic decomposition of methionine. This is confirmed by the high recovery of sulfide in both the gas and in the sludge with the cystine additions. There was less increase in the sulfide content of the sludge with the methionine additions, and sulfide was found in the gas with only the 1,000 p.p.m. addition.

The interference with digestion caused by the cystine appears to be the result of the sulfide formed in the breakdown of this compound. The interference is similar to that found with soluble sulfide additions, in that gasification is inhibited, and volatile acids accumulated.

The addition of the methionine actually stimulates digestion. The volatile acids were greatly increased with all additions, and gasification was markedly increased with the 100 p.p.m. addition. Although the volume of gas per gram of volatile matter added was increased, the solids and volatile matter reductions were not increased. Greater methionine additions resulted in less solids and volatile matter reduction. Some sulfides were found with the 1,000 p.p.m. addition. This

may have been the cause of the decrease in solids and volatile matter reduction. Also, the high volatile acids concentration may have interfered with gasification.

It, therefore, appears the effect upon sewage sludge digestion of an organic sulfur containing compound is dependent upon the breakdown of that compound into soluble sulfides which have an inhibiting effect. Also, methionine in the range of 100 p.p.m. as sulfur appears to stimulate sewage sludge digestion.

Discussion

The sulfide ion may greatly interfere with sewage sludge digestion when present in a soluble form. The sulfide interferes with gasification and the utilization of volatile acids. The buildup of volatile acids also further interferes with gasification. Therefore, the alkalinity or acidity of the sulfide and the sludge under digestion are also factors affecting digestion. Any alkalinity present will tend to neutralize the volatile acids, with the result that the effect of the volatile acids will be lessened or eliminated. Neutral soluble sulfide causes a definite retarding effect upon digestion at about 100 p.p.m.

Sulfide is a particular problem in sewage sludge digestion because the process is an anaerobic one, and any partly or completely oxidized inorganic sulfur compounds present may be reduced to sulfide. Sulfite is one of the compounds that might be reduced to sulfide. Sulfates are also reduced to sulfides. Many organic compounds containing sulfur produce sulfide as one of the products of decomposition. Cystine has a greater tendency to produce sulfide when broken down than does methionine.

Sulfite is also toxic in itself, as shown by the immediate suppression of gasification. The sulfite then is reduced to thiosulfate and further to sulfide during the digestion, thereby interfering with digestion in two ways.

When gasification was inhibited by sulfide the volatile acids first accumulated and then decreased. On the other hand, with the addition of 100 p.p.m. methionine-S, volatile acids and gasification both increased without any increase in solids and volatile matter reduction.

Summary

A study was made of the effects of several sulfur containing compounds on sewage sludge digestion. The sulfur compounds used were: sulfite in concentrations up to 250 p.p.m. as sulfur; neutral soluble sulfide in concentrations up to 250 p.p.m.; several sulfides of varying solubility and reaction; and organic sulfur in the forms of cystine and methionine in concentrations up to 1,000 p.p.m. as sulfur. For each experiment, the results were compared with a control containing no added sulfur compounds, and conclusions were based upon these comparisons.

Conclusions

In a digester, sulfite is reduced to thiosulfate, and then to sulfide. However, the sulfite itself is toxic to digestion. The presence of even 50 p.p.m. sulfite sulfur retarded digestion, and the presence of 100 p.p.m. almost completely inhibited digestion. The sulfide formed by the reduction of the sulfite further retards digestion. The action of the sulfite appears to be the retardation of gasification more than liquefaction, resulting in an accumulation of volatile acids which may slow digestion even more.

The effect of any sulfide on sewage sludge digestion depends upon four main factors: (a) the concentration of the sulfide salt; (b) the solubility of the salt; (c) the concentration of sulfide in the salt; and (d) the acidity of the salt. The greater these four factors, the greater will be the interference with digestion. Neutral, soluble sulfide interferes with digestion in the

range of 100 p.p.m. This interference appears to be manifested in two ways: (a) the sulfide has a direct inhibitory effect upon the organisms responsible for gasification; and (b) by reducing gasification volatile acids which accumulate and subsequently lower the pH have a greater inhibitory effect upon gasification. The higher the sulfide concentration, the lower the volatile acids concentration which further retards digestion. However, increasing the sulfide concentration (to about 150 p.p.m.) also shows a tendency to retard liquefaction.

The interference with sewage sludge digestion caused by the presence of an organic sulfur containing compound which is not toxic in itself, is dependent upon the production of sulfide from that compound. Neither cystine nor methionine was found to be toxic. However, cystine despite its low solubility of only 27 p.p.m. was found to breakdown into sulfide, with the result that 100 p.p.m. as sulfur retarded digestion, and 500 p.p.m. completely inhibited gasification. Methionine is only slightly broken down into sulfide, resulting in little retardation of digestion with even 1,000 p.p.m. additions as sulfur. Methionine actually stimulated the volatile acids production with the result that more gas was produced with the addition of 100 p.p.m. than in the absence of methionine. However, this concentration of methionine did not increase the solids and volatile matter reduction.

Acknowledgment

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Industrial Wastes

BROAD FIELD DISPOSAL OF BEET SUGAR WASTES

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The early history of beet sugar is clouded in the vastness of antiquity. It is believed that the Chinese have made a crude sugar from beet juice since very early days (1). The opening of a small factory at Breslau, Germany in 1801 was the forerunner of the gigantic European beet sugar industry.

Cultivation of sugar beets was introduced into the United States in 1880. By 1948, 85 plants operating in 16 states produced 1,369,000 tons of beet sugar from 9,422,000 tons of beets; this production represented 12.3 per cent of the world total. Associated with beet sugar manufacturing is the necessity for a copious supply of water and the related problem of waste disposal. For each ton of beets processed, 2,500 to 3,500 gal. of water are used, and plant capacity varies from 1,000 to 5,900 tons of beets per day. It has been estimated that, during the season of operation, the beet sugar industry in the United States discharges liquid wastes having a biochemical oxygen demand potential of nearly 3,000,000 lb. daily (2).

It is to the credit of the beet sugar industry that efforts are being made to correct stream pollution problems caused by large discharges of untreated wastes. The Great Western Sugar Company has studied various waste disposal systems, has utilized several methods for treatment of its wastes, and at Bayard, Nebraska, is now evaluating the application of wastes to a grassland field prior to their ultimate drainage to a watercourse. This study was undertaken to assist in the understanding of the latter treatment process. It is typical of the cooperative studies being encouraged by the National Technical Task Committee on Industrial Wastes.

Development of Cooperative Study

A work plan drafted by the Missouri Drainage Basin Office, U. S. Public Health Service, was reviewed by the Nebraska Department of Health, the Great Western Sugar Company, the Division of Water Pollution Control, and the Robert A. Taft Sanitary Engineering Center of the Public Health Service. A six-man survey team was assigned to Bayard, Nebraska, for one month starting October 19, 1953, to make laboratory analyses and secure other basic data. The team consisted of a sanitary engineer, a chemist, a bacteriologist, and three sample collectors.

The aims of this study are briefly outlined as follows:

1. To evaluate the over-all effectiveness of grassland field treatment of beet sugar wastes.
2. To evaluate treatment accomplished in various segments of the field.
3. To determine the effect of the field effluent on the receiving drain ditch and on the North Platte River.

4. To record bacteriological and limnological trends through the treatment area.

Plant Operation

Products of the Bayard plant are sugar, molasses and dried pulp. Raw materials include sugar beets, lime rock, sulfur, soda ash, coke and coal. The plant has a rated capacity of 1,850 tons of beets per day. During the 1952–53 season, the average daily cut was 1,769 tons with a maximum of 1,983 tons.

Beet sugar plant operation is continuous, barring accidental stoppage, from the start in the fall until all beets are processed. The 1952 operation began about October 11 and lasted for 89 days. The Bayard plant operates as a non-Steffen House (straight) sugar factory.

A flow diagram is illustrated in Figure 1. The beets are conveyed to the processing plant where leaves, floating debris, sand and beet rootlets are removed by screening as the beets are washed in the flume and washers. The washed beets are sliced into long, narrow, V-shaped ribbons termed "cossettes." Conveyor belts transport the cossettes to diffusers for the extraction of sugar by counter-flow with hot water. The raw juice extracted is subjected to chemical treatment and filtration following which it is concentrated in multiple-effect evaporators and finally is crystallized in vacuum pans. The residual molasses is shipped to a Steffen House plant for additional sugar recovery or for by-product recovery. The spent cossettes are drained, then further dewatered by pressing, and finally dried and bagged as high grade stock food.

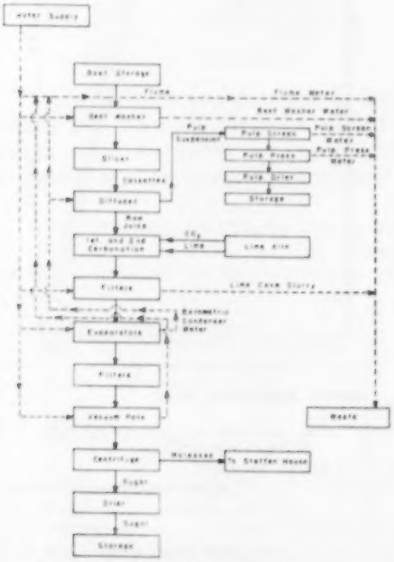

FIGURE 1.—Beet sugar factory flow diagram, Great Western Sugar Co., Bayard, Nebr.

Water Supply

Process water is diverted from an irrigation ditch; approximately 43,200,000 lb. or about 5.2 m.g.d. is the average usage.

Wastes

Flume and Wash Water

Flume and wash water, which amounts to about 75 per cent of the total water used at the plant, is that used to convey beets from storage piles and sheds to the factory. Principal pollutants are leaves, beet particles, beet rootlets and dirt washed from the beets. An average of 30 lb. of dirt may be removed per ton of beets; wet weather conditions will increase this amount by over 50 per cent. Organic material and sugar dissolved from spoiled or frozen beets may contribute additional polluting matter. The flume and wash water make-up includes fresh water and waste from barometric condensers. The condenser water contains dissolved organic vapors and a small amount of ammonia.

Pulp Screen Water

Much of the water remaining in the beet slices (cossettes) after extraction of sugar is removed from the spent material by screening. The pulp screen

water is considerably less in volume than flume water, but carries a greater concentration of organic materials as dissolved sugar, protein, salts, pectin, cellulose, pentosans, and other substances.

Pulp Press Water

Before the screened pulp enters the dryer, it is pressed to remove as much liquid as possible. The plup press water is discharged to waste and the dried pulp is used for cattle feed.

Lime Cake Wastes

Lime cake is the precipitate filtered out of the lime-treated and carbonated juice. The cake, diluted with water, is pumped as a slurry to the main plant sewer. On a dry basis, the lime cake will average about 50 lb. per ton of beets sliced. This waste contains, in addition to the calcium carbonate, considerable organic matter as sugar, organic nitrogen, phosphates and other compounds.

Miscellaneous Wastes

Floor washings and washings from filter cloths are returned for reprocessing if they contain sugar, otherwise they are wasted to the sewer.

Waste Disposal Practices

Receiving Stream

The irrigation ditch passing through the plant property is the ultimate recipient of the treated and untreated plant wastes. The ditch parallels the main plant sewer canal and then runs adjacent to and east of the treatment field. The field and drain ditch are separated by a dike for the full length of the field, as illustrated in Figure 2. A collection channel at the lower end of the treatment field discharges to the ditch which flows into the North Platte River, about one mile distant from this point. The drain is a clear water stream supporting fish life and having year around flow.

Sewerage

Two sewers convey the wastes from the factory. One, a 12-in. tile line, empties directly to the drain ditch about 75 ft. below the process water supply diversion dam. The other line, a 27-in. sewer originally discharging to the drain ditch, now empties to a waste canal which conveys the waste to a pump house and thence to the treatment field. When the pump is down

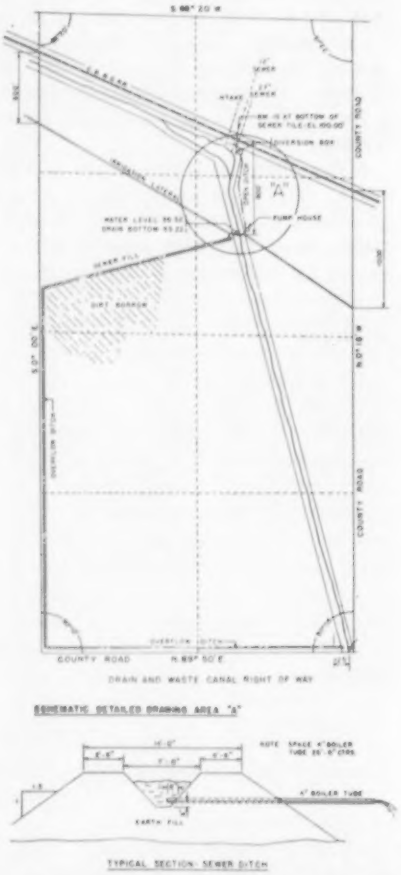

FIGURE 2.—Plan of ungraded natural grassland field and ditches used in the broad field treatment of sugar beet wastes.

for maintenance, the main factory wastes are bypassed directly to the drain ditch.

Natural Grassland Treatment Field

The treatment field consists of 160 acres, as illustrated in Figure 2. It is roughly rectangular in shape, 3,500-ft. long and 1,750 ft. wide at the inlet end. Waste is applied to the field by means of a distribution flume having a dirt bottom and wood sides; the outlets from the flume to the field are 4-in. pipes spaced at 25 ft. intervals.

The natural grassland field, essentially ungraded, is generally level but not level enough to prevent channeling. Only a limited portion of it could be truly called a "grassland filter," with the waste passing through the grass in shallow sheets. The lack of uniform distribution reduced the effective area and permitted excessive short circuiting through the field. As the natural flow of the waste over the field reaches the lower end, drainage is collected in the effluent canal, and is discharged to the drain ditch.

Sampling Procedure

The drain ditch was sampled at the factory water supply diversion to provide a reference point for evaluation of the effects of untreated factory waste and treatment field effluent upon the ditch as detected by the series of downstream stations. The main factory wastes were sampled at the distribution header prior to waste application to the treatment field. The treated effluent sample was collected at the discharge flume.

To control the flow studies, the treatment field was staked in 200-ft. squares. By use of a coordinate system, personnel studying the field and observing the passage of dye were able to orient themselves and to locate and record exact positions. River samples were usually taken from the main channels and composited; dissolved oxygen and coliform concentrations were determined on the individual grab samples.

Examination and analyses were made in accordance with "Standard Methods for the Examination of Water and Sewage," Ninth Edition, 1946.

Presentation of Data

Plant Wastes

The two sewers discharging wastes were gaged and the following results obtained for the period of study. The 12-in. tile line that emptied to the ditch had an average discharge of 1.6 c.f.s. with a B.O.D. value of 337 p.p.m. and contained 1,056 p.p.m. suspended solids. The main plant out-fall had an aver-

TABLE I.—Results Showing Average Values in P.P.M. and Pounds Concentrations of Various Parameters

Station		Flow	D.O.		B.O.D.		Suspended Solids	
		c.f.s.	p.p.m.	lb.	p.p.m.	lb.	p.p.m.	lb.
D–1	Ditch, below water supply intake	31.1[1]	9.89	1,660	2.37	398	70	11,750
S–1	Sewer to ditch	1.6	1.29	11	337	2,910	1,056	9,130
D–2	Ditch immediately above field	32.7[1]	9.31	1,640	32.3	5,700	—	—
F–1	Factory effluent to field	4.3	0.97	22	483	11,200	5,215	121,900
D–3	Ditch before receiving effluent	32.7[1]	7.96	1,405	>37.1	>6,550	92	16,242
F–5	Field effluent	3.5	0.44	8	158	2,990	63	1,190
D–4	Ditch downstream	37.0	5.74	1,147	52.4	10,470	166	33,167

[1] Estimated flow.

age flow of 4.3 c.f.s., as measured at the field distribution flume, with a 483 p.p.m. B.O.D. value and contained 5,215 p.p.m. suspended solids. The B.O.D. in these two combined discharges is 14,110 lb. per day (Table I).

Approximately 2,400 lb. of B.O.D. reached the drain ditch by overflow or seepage where the waste canal parallels the ditch (as ascertained by ditch sampling) and must be added to the previous measured values. Thus the B.O.D. of the waste from the plant approximated 16,500 lb. per day. Based on an average daily cut of 1,890 tons of beets, the B.O.D. in pounds per ton of beets was 8.7. The average daily suspended solids load as discharged by the two sewers was 131,000 lb. of which 121,900 lb. were in the main sewer discharging to the treatment field.

Treatment by the Grassland Field

The average B.O.D. value of the influent was 483 p.p.m. and of the effluent 158 p.p.m., a reduction of 67 per cent. Suspended solids were reduced 99 per cent from 5,215 p.p.m. to 63 p.p.m. The over-all efficiency of the treatment plot might be better illustrated by reference to Table I showing pounds value. It will be noted that the average volume of the influent applied to the field during the period of study was 4.3 c.f.s. while the effluent was 3.5 c.f.s. Of the 11,200 lb. of B.O.D. applied daily to the field 2,990 lb. were discharged in the effluent, a total reduction in B.O.D. of over 73 per cent.

It is pertinent to consider results of the sectional field sampling stations. The B.O.D. data reveal that at 300 ft. down field, no appreciable reduction took place; at 450 ft., the reduction was 48 per cent; at 600 ft., the data indicated a slight increase; and at 900 ft. the removal was 60 per cent.

Other parameters depict a like picture. Treatment appears to accomplish primarily the removal of settleable solids in the upper one-eighth of the field with subsequent biological removal of about 10 per cent of the dissolved solids as the waste passed over the rest of the field.

The bacteriological results in the field were erratic, as anticipated. Prior to the campaign the field was used for grazing by cattle and numerous prairie dogs inhabited the tract. Fecal matter from these animals no doubt contaminated the ground. The coliform results varied considerably, yet the over-all median reduction was recorded at 89 per cent. The total 37° C. bacteria count showed a constant downward trend with an over-all reduction of 75 per cent.

Biological Features

Aquatic life in the treatment field during November 1–3, 1953, consisted mainly of the sewage fungus, *Sphaerotilus*, and a unicellular green alga, *Chlamydomonas*. The fungus was attached to grass stems and leaves, mainly in the flow channels, and large areas were blanketed with its long grayish streamers. The algal growths occurred as free swimming forms in suspension, but their greatest concentrations were found adhering to grass and other bottom materials in quieter areas, and in a few instances to filaments of fungus.

Chemical analyses showed oxygen to be present in small amounts (1.6 to 2.4 p.p.m.) only in the clear brown water (quiescent areas). Absence of oxygen in the green areas was apparently the result of immediate exhaustion of quantities produced by photosynthesis, as the algae concerned did not appear to be existing as saprophytes. Nitrogen cycle analyses of water collected in various areas showed greatest concentration of ammonia and nitrate nitrogen (4 p.p.m. NH_3—N, 0.05 p.p.m. NO_3—N) in areas containing an algal growth. Nitrate content of water in the ditch above the area was far greater than that found anywhere on the dis-

posal area (1.5 p.p.m. NO_3—N). Nitrite nitrogen occurred in the ditch but was absent in water on the field.

Concentration of algae in suspension over different areas of the field varied widely. The effluent from the area contained an average concentration of 66,500 *Chlamydomonas* per ml. Water over the green algal areas contained 775,000 to 1,650,000 *Chlamydomonas* per ml.; grayish water flowing around algal areas and over fungus areas possessed 64,000 *Chlamydomonas* per ml.; and the areas of clear brownish water contained no suspended organisms. Concentration of algae adhering to the substratum was much greater than that of suspended individuals, and was estimated to amount to approximately 23,000,000 per square centimeter of surface in green areas, an extremely high concentration.

Examination of plankton in the water supply ditch disclosed a varied algal population, mostly displaced benthic diatoms. To this assemblage was added *Chlamydomonas* from the treatment area, providing an average concentration of 337 per ml. As dilution in the ditch was roughly 10 to 1, this concentration was about $\frac{1}{20}$ of what would be expected to result from dilution alone. One sample taken about 150 ft. below the treatment area outfall contained 5,003 *Chlamydomonas* per ml. These facts indicated a rapid die-off of this green alga in the ditch.

Above the entrance of wastes from the factory, the ditch contained dense attached growths of *Cladophora*, a filamentous green alga, along the strands of which were attached many diatoms. Photosynthesis by these algae resulted in supersaturated oxygen levels during daylight hours; and algal respiration continuing after cessation of photosynthesis at dark resulted in oxygen decline during the night. Oxygen saturation noted early one afternoon was approximately 125 per cent. During the night (early a.m.) oxygen declined to less than 90 per cent saturation.

Experiments With Algal Growths

Large samples of field effluent were collected and transported to Kansas City, where they were transferred to glass pans. Depth of effluent in the pans varied from $1\frac{1}{2}$ to 4 in. Chemical and biological analyses of water and bottom growths were conducted at intervals for 26 days. Water temperature varied from 20° to 21° C. over this period.

At the start, a supersaturated oxygen condition (142 per cent) was evident in all pans, and the algal growth which had at first consisted entirely of *Chlamydomonas*, had become quite varied. *Chlamydomonas* was evidently on the decline and diatoms of the genera *Nitzchia* and *Navicula*, as well as two green algae (*Ankistrodesmus* and *Chlorella*) and *Euglena* were increasing rapidly. This trend continued. After 16 days, *Chlamydomonas* was relatively rare and diatoms and the two green algae listed above were by far predominant. *Euglena* had also continued to increase and was about three times as numerous as *Chlamydomonas*.

Monocarbonate alkalinity eventually increased to 108 p.p.m. and bicarbonate alkalinity declined from an original 412 p.p.m. to 236 p.p.m. In relation to these alkalinity changes, pH reached a maximum of 9.3. Ammonia nitrogen increased from 0.45 to 1.65 p.p.m.; nitrite was never present; and nitrate nitrogen increased from 0.02 to 0.2 p.p.m. Phosphorous was never evident in the available or soluble form. Supersaturated oxygen levels were evident at all times of analyses, which were restricted to daylight hours.

Discussion of Biological Features

Although the algal growths scattered over the treatment area were not observed to add any oxygen that persisted (on even a temporary basis), the growth of various forms of life must have been at the expense of materials

in the sugar beet wastes, which were utilized in formation of protoplasm as well as in metabolic processes. Concentrations of the fungus growth were not estimated, but the algae adhering to the substrate were densely developed. The protozoa and their bacterial food organisms developed also at the expense of materials contained in the waste water. Elements required to develop and maintain varied forms of life were taken from the wastes. This additional purification, as measured by B.O.D., seemingly approached 10 per cent of the original concentration.

By virtue of their bacterial and algal content, green areas were apparently more effective in removing dissolved substances than the sections containing *Sphaerotilus*, although the latter were undoubtedly involved in carbohydrate removal. Restriction of green algal areas to rather scattered sections was apparently a result of shifting of flow paths and short circuiting in passage across the field. Establishment of a uniform course of flow involving most of the area, and a rather uniform shallow depth, would greatly enhance conditions for algal and other growths and probably result in additional reduction of waste compounds.

Effect on Receiving Waterways

The drain ditch water, at the location where the factory supply is withdrawn, had an average of 9.89 p.p.m. D.O., a B.O.D. value of 2.37 p.p.m., and a median coliform M.P.N. of 2,300 per 100 ml. About 200 yd. below the 12-in. sewer outfall (which discharges untreated wastes), the D.O. dropped, and the B.O.D. increased. These data are given in Table I, for stations D–1, D–2, D–3, and D–4. The coliform content also increased. The minimum recorded D.O. value in the ditch water was 4.19, which is below the 5 p.p.m. level established as desirable for non-trout waters in the Missouri Basin (3). It is probable, however, that discharge of all plant wastes untreated to the ditch would have caused a situation approaching nuisance conditions in the lower reaches of the ditch.

The results of river sampling (grab samples) compared reasonably well with a study of the North Platte River made in 1950 (4).

Grab Versus Composite Samples

The fact that beet sugar factory operation is generally continuous has led to the assumption by many that grab samples would adequately provide representative results. A limited evaluation of this factor was made, particularly at stations F1 and F5 (Table I). Generally, average, and median values also, of the two groups were reasonably comparable where a considerable number of a test were conducted. The maximum and minimum values of the grab samples showed greater extremes. The longer the period of compositing, the less is the variation from the average or median value. For example, the B.O.D. results of F1 are quite typical:

B.O.D. (p.p.m.)	Sample		
	Grab[1]	Composite	
		8–10 hr.[2]	24 hr.[3]
Maximum	1,083	746	495
Average	496	493	432
Median	423	472	426
Minimum	208	252	375

[1] 15 samples.
[2] 13 samples.
[3] 3 samples.

It would appear that for occasional sampling, 24-hr. composites are the most reliable.

Odor Conditions

The objectionable odor occasionally associated with sugar beet plant operation invariably may be traced to unsuitable waste disposal practices. The prevention or reduction of these odors has been a goal of plant management.

Observation during the period of study indicated a relatively low level of atmospheric odors except in close proximity to the field, a factor favoring the disposal method practiced at the Bayard sugar factory.

Treatment Field Flow Characteristics

The flow pattern of the field was determined by observation and by flow studies using dye and salt. Congo red dye was first chosen as the flow indicator inasmuch as it was a contrasting color to the green algae sections found in the middle portion of the field. However, the red dye faded out completely in the first 500 ft. of the field, due either to settling out with the lime slurry or to excessive dilution. Fluorescein dye was then added to the field influent and traced through the field until it was lost or masked by the similarity in color of the algae-laden waters.

To determine the over-all detention time of the field, 1,000 lb. of cattle salt (sodium chloride) were added at the pump intake just prior to station F1 and hourly samples collected for 50 hr. at the field effluent station F5 (Table I). The results of the chloride study are shown in Figure 3.

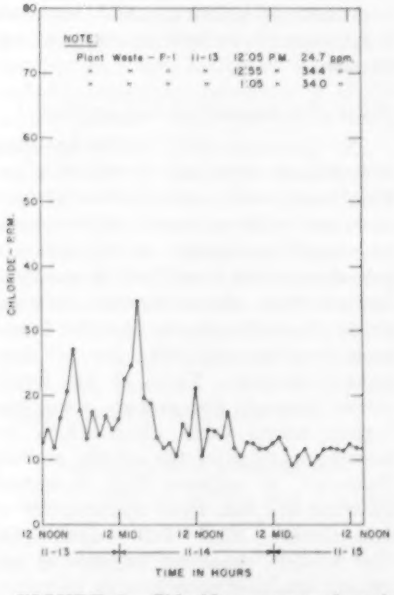

FIGURE 3.—Chloride concentrations in grassland effluent at station F-5 after dosing sugar beet waste with 1,000 lb. of salt.

The maximum concentration of salt essentially reached the field at about 1:00 PM, a considerable portion passed through the field in 4 hr., the maximum concentration reached the effluent in 14 hr. and most of the salt cleared the field within 29 hr. The average detention time of waste through the field would then be one-half of 29 hr. or 14.5 hr. which coincided well with the maximum concentration detected in 14 hr.

Discussion

Waste Evaluation

The principal wastes from the Bayard plant emanated from fluming and washing of beets, pulp screening, pulp pressing and flushing of the lime cake. The B.O.D. value of the waste from the plant during the period of study was 16,500 lb. per day, which is considered low. The summer and fall were relatively dry and warm in the Platte valley. The harvesting of beets proceeded rapidly with a minimum of adhering soil and no freezing. Thus the beets arrived for processing in excellent condition and it would be expected that the wastes would be relatively low.

It has been reported (2), in a study of waste similar to the waste discharged at Bayard, that the B.O.D. of the lime cake slurry would amount to 56 per cent of the total B.O.D. discharged. A 1952 unpublished report (5) by Great Western Sugar Company on the Bayard plant indicated a similar percentage. Other studies show a somewhat lower percentage (6) although recognizing the relatively large portion of this waste on a B.O.D. basis. Because of the polluting material in the lime waste, other possible methods of lime cake disposal should be given serious

consideration in any over-all waste disposal problem. At Bayard, the beet sugar company decided that economics favored hydraulic removal along with other plant wastes.

Waste Volume Study

It is interesting to endeavor to account for the volume of the liquid wastes as it proceeds from the plant through the waste treatment field. Flow through the field during period of study was reduced from 4.3 c.f.s. to 3.5 c.f.s. or 0.8 c.f.s. (19 per cent). Several factors causing this reduction are evaporation, percolation, transpiration and leakage. Mean measured evaporation was 0.10 in. per day during period of study. According to the U. S. Weather Bureau, the standard evaporation pan factor is 0.7 when applied to broad water surfaces. Assuming that 60 per cent of the 160 acre field was wetted (96 acres), the evaporation was 0.56 acre-ft. or 0.28 c.f.s. One reason for the low evaporation rate was the unusual calm weather prevailing during this study. Evaporation varies with wind velocity as shown by the following from Merriman-Wiggin's "American Civil Engineers Handbook" (7):

Wind velocity (m.p.h.)	0.0	5.0	10.0	15.0	20.0
Relative evaporation	1.0	2.2	3.8	4.9	5.7

The wind velocity, measured at Bridgeport, Nebr. showed a median of 1.46 miles per hour; the maximum was only 4.71 mi.

Transpiration appears to be a negligible factor, as only six per cent of the total water used by plant consumption occurs in October and pasture growth consumption may average only 0.5 ft. per year (7).

Percolation losses on irrigated lands may vary from 20 to 75 per cent of the water applied, increasing with soil porosity. Soils of the North Platte valley are adaptable to irrigation and it appears reasonable to expect appreciable percolation unless extensive soil clogging occurs. Recorders were installed and operated 55 days until cold weather when ice formation prompted their removal. The inflow totaled 437 acre-ft. and the outflow, 343 acre-ft.; the volume reduction was 21 per cent. Flow for the period showed the average influent as 4.1 c.f.s. and effluent as 3.8 c.f.s. Evaporation could account for 0.28 c.f.s. until cold weather and freezing occurred. The closeness of the values would indicate clogging of soil with a minimum of percolation. Numerous prairie dogs living and burrowing over scattered parts of the treatment field indicated the ground was not saturated with water. Occasional leakage was observed from the field to the drain ditch by overtopping and erosion of the dike or through prairie dog burrows. To prevent the adverse effects of insufficiently treated wastes on the receiving water course, care must be exercised to construct and maintain dikes so as to preclude overtopping and breaks.

Field Maintenance and Operation

The grassland field, called by some a grassland filter and by others a settling basin, combined these two actions. A waste with a considerable amount of suspended matter and much dissolved material is applied to the field. Immediately, the settleable material drops from the liquid, and the liquid soon becomes relatively free of suspended matter. Then, as the liquid filters through the grasses, some biological action takes place which removes a portion of the soluble matter. However, it appears that biological growths did not have opportunity to demonstrate their full capabilities. The median dissolved oxygen in the effluent was zero.

There are many factors that would improve operation and treatment re-

sults of the field. The waste reaching the distribution flume contained beet tops, rootlets and other coarse matter that should be screened out at the plant to prevent operational difficulties in the flume. Flow from the distribution flume to the field was seldom uniform because of the intermittent operation of the lift pump and clogging of the pipes leading from the distribution canal. Rectification of these defects could readily be accomplished by a suitable pump installation and by the use of open distribution flumes leading to the field from the distribution header. This latter correction was being installed during this study.

The effectiveness of the grassland field could be improved by leveling and terracing in the manner of the border strip method of irrigation, thereby increasing the detention time through the field. If leveling and terracing would permit 80 per cent of the field to be usable with liquor depth of 3 in., the average calculated detention time would be 72 hr., or about 5 times the present detention period. This additional time would allow use of a goodly portion of the field as a true biological filter with added purification, particularly of the soluble fraction that is not now being removed by settling.

Field treatment appears to offer considerable flexibility. Where a lesser degree of treatment is required, a much smaller plot to provide settling only might suffice. Where land is scarce, a good deal of the solids could be prevented from reaching the grassland field by separate disposal of filtered lime cake, and the remainder of the waste treated on a smaller field.

Summary and Conclusions

During this study, the liquid wastes amounted to more than 2,400 gal. per ton of beets processed and had a B.O.D. value of approximately 16,500 lb. per day. Sixty-eight per cent of the B.O.D. produced was applied to a natural grassland field for treatment while the remainder reached the ditch untreated.

Of the 4.3 c.f.s. with an average of 483 p.p.m. B.O.D. applied to the field, 3.5 c.f.s. with an average B.O.D. value of 158 p.p.m. was returned to the drain ditch as effluent from the field after an average of 14.5 hr. detention in the treatment plot. The B.O.D. removal in parts per million was 67 per cent, while over 73 per cent of the pounds B.O.D. applied remained on the field. Suspended solids removal exceeded 99 per cent.

Bacteriological reduction through the field was 89 per cent based on M.P.N., and 75 per cent total based on bacteria count at 37° C. Algal and protozoan growths were present in large numbers especially in the quiescent sections of the field.

Effluent from the treatment field, waste discharged directly to the drain ditch, and inadvertent spillage or overflow of untreated waste and partially settled liquor from the treatment plot increased the B.O.D. in the drain ditch from 2.37 p.p.m. to 52.4 p.p.m. while the D.O. dropped from 9.89 p.p.m. to 5.74 p.p.m.

All plant wastes should be applied to the treatment field and closely controlled to prevent inadvertent spillage or overflow. More equable distribution of the waste to the field would result in increased efficiency of the treatment area. Field leveling and terracing should be undertaken to increase effective area and improve detention time, so that advantage might be derived from additional biological activity.

The broad application of sugar beet wastes to a natural grassland plot appears to be an effective method of waste treatment.

Acknowledgment

This study was made possible by the excellent cooperation of the Nebraska and Wyoming state boards of health, the Nebraska Department of Roads and Irrigation, the Great Western Sugar

Company, the U. S. Weather Bureau and the U. S. Geological Survey in furnishing personnel, equipment, or other facilities necessary to the success of this survey.

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PROFESSOR LEON B. REYNOLDS DIES

The first president of the California Sewage and Industrial Wastes Association, Leon B. Reynolds, died at Palo Alto, California, on September 26, 1955. He was professor of hydraulic and sanitary engineering at Stanford University and a specialist in sewage and water works developments. A graduate of Stanford in 1909, he worked with Burns and McDonnell, consulting engineers, until 1923 when he returned to teach at Stanford.

Professor Reynolds helped organize the California Member Association and served as president in 1928.

ANAEROBIC DIGESTION OF PEA-BLANCHER WASTE

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The processing of peas for canning results in a large volume of combined wastes. These wastes exert a high biochemical oxygen demand (B.O.D.) which must be satisfied by treatment processes or by natural purification in a receiving body of water. Reduction of the B.O.D. by lagooning is not always satisfactory because of: (a) difficulty in controlling the process; (b) adverse topographical and geological features of the area; or (c) proximity to habitations.

The pea cannery often is located in a small town in a predominantly agricultural district. Municipal sewage treatment facilities either may not be available or are not designed to accommodate the greatly increased waste load during the relatively short canning season. Many canneries do not have adequate provisions for separate treatment or disposal of waste, frequently because of the absence of a watercourse that has sufficient flow to absorb the pollution load.

Table I shows that the pea canning wastes from the blancher and blancher spray reels constitute about 7 per cent of the process waste flow, but exert approximately 68.6 per cent of the total B.O.D. (1). The separate treatment of this small portion of the waste would result in a marked decrease in the B.O.D. of the remaining cannery discharge.

Literature Review

The formation of methane in nature has been known for some time and has been recognized as a product of biological activity. A relation between inflammable gas and putrefaction was recorded by Volta (2) in 1775. Popoff (3) in 1875 conducted experiments to determine the source of methane, after Reiset (6) in 1856 noted the formation of methane from decomposing manure.

Anaerobic treatment of sewage and wastes has been utilized since about 1881 (4). Buswell (5) cites examples of industrial scale anaerobic digestion applied to molasses waste resulting from the production of yeast, and gives examples of the same process used for the reduction of waste derived from grain in the manufacture of butanol and ethanol.

The gas evolved from the anaerobic process is composed largely of carbon dioxide and methane. The carbon dioxide and methane amount to about 95 per cent of the total volume. The methane to carbon dioxide ratio varies from 1:1 to 3:1 (7) (8) (9) (10) (11) (12) (13) depending upon the composition of the substrate and the conditions under which the fermentation takes place. Relatively simple substances such as sugars, some proteins, and fats may be completely gasified, but when the original substrate is composed of more woody material the humus-like sludge remaining may amount to 40 per cent of the original volume of solids (8) (14) (15) (16) (17).

Maintenance of efficiency (19) (20) has been a major problem in the operation of anaerobic digestion units controlled by feed rates which do not allow the volatile acids concentration to rise above a predetermined value (21) (22) (23) (29). Stander (18) reported on a laboratory investigation

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TABLE I.—Pollutional Characteristics of Pea Cannery Wastes

Waste	Rate of Flow			Five-Day 20° C. B.O.D.			
	g.p.m.	Total (%)	Process (%)	p.p.m.	Pounds per Hour of Flow	Total (%)	Process (%)
To storm sewer	292	26.3	—	143	21	2.6	—
Process waste	819	73.7	100	1,900	777	97.3	100
Blancher overflow	18	1.6	2.2	35,000	315	39.4	40.5
Blancher spray reels	39	3.5	4.8	11,150	218	27.3	28.1
Other	762	68.6	93	640	244	30.6	31.4

of the anaerobic digestion of wine-pot still residue and yeast waste. The results indicated that any factor which could cause an unbalance in the complex bacterial activity affected the efficiency of the purification process, and that a steep rise in volatile acids production was to be regarded as the first indication of such an unbalance. The organic matter in a fermentation effluent differs in concentration and type from that in raw sewage sludge. Investigators generally have given consideration to the difference in concentration, but not to the possible effects that the different nature of the organic matter might have on the efficiency of anaerobic digestion.

The organic and mineral components of domestic sewage sludge seem to supply all the requirements necessary to maintaining the state of balance (24) in the complex association of bacteria present in a sewage sludge digester—provided that physical conditions are suitable. In the case of organic wastes other than sewage, the nature of the organic matter will probably constitute a more important factor than concentration in causing an unbalance in the bacterial population.

The sludge in an active digester provides favorable environmental conditions (25). The initial slow rate of digestion and of methane production in a new digester are conditions not due to paucity of methane-producing bacteria, but to the unsuitable environment. Wastes derived from operations such as the blanching of peas, however, are sterile or contain only a few specific organisms. No fresh inoculum of sewage sludge bacteria, necessary to the anaerobic fermentation, is added. Also, the constant withdrawal of digester effluent removes a portion of the bacteria originally present in the digester. It has been suggested that anaerobic digestors used for wastes other than sewage sludge could be maintained in a healthy condition, not by reducing the feed rates as the volatile acids content increases, but by the addition of a specially prepared inoculum to remedy the state of bacterial unbalance as indicated by accelerated accumulation of volatile acids (18).

The environment necessary for satisfactory anaerobic digestion has been delineated as a result of laboratory work and experience with pilot-plant and full-scale digesters. While knowledge of the microbiology and biochemistry of a normal sewage plant is quite limited, Mylroie (26) has developed a culture method by which *Methanobacterium formicicum* can be isolated and its numbers in sludge determined. Since methane-producing organisms are incapable of digesting complex organic matter, auxiliary micro-flora (26) capable of hydrolyzing the organic material must be associated in the destruction of most wastes.

Anaerobic Digestion Requirements

The conditions required for successful reduction of B.O.D. by anaerobic digestion may be defined. After the

break-in period an anaerobic digester has an active biota acclimated to the specific environment. Organisms capable of adapting themselves to the conditions have survived. It is possible that some active organisms are mutants that have propagated because of a favorable environment. Any sudden change in environmental conditions will result in a drop in efficiency.

Mixing is essential to successful anaerobic digestion. Thorough mixing brings the added waste into contact with the organisms, increasing bacterial use of the substrate and releasing entrapped gas. Mixing aids in maintaining uniform temperatures, and also eliminates undesirable stagnant pockets. Mixing has been shown to produce a 26 per cent increase in gas production, and to improve B.O.D. reduction by 30 per cent, in one specific case in the laboratory (27).

A method of sewage sludge digestion recently has been developed which makes it possible to complete the anaerobic reduction of sewage solids in approximately one-third the usual time (28).

Uniformity of temperature is stated to be very important in that significant changes will result in a decrease in bacterial activity (4) (18). Mesophilic temperature ranges have been used successfully in many instances. Research has shown these temperatures to be favorable to methanogenic organisms (26).

Little methane will be produced when the pH is below 6.2 or above 8.8. The optimum pH for the methane producing bacteria is 6.8 to 7.2 (4). It has been shown (29) that the addition of alkali may induce a highly ionized condition of the substrate which will tend to cause the formation of acetate rather than acid from the organic material.

There is no generally accepted optimum feed rate for anaerobic fermentation of organic wastes. The method of Adams and Hungate (30) for the prediction of continuous fermentation cycle times from growth curve analysis was applied (31) to pea-blancher waste. The computation gave an optimum cycle time of 9 days. This method of predicting the length of a cycle is based on the fact that, in a continuous fermentation, an equilibrium is established in which the concentrations of nutrients, substrate and metabolic products are maintained constant: (a) by the addition of fresh medium, and (b) by the removal of an equal volume of fermented or partially fermented medium. The position of this equilibrium defines the degree of fermentation. The rate of waste feed is calculated to equal the rate of bacterial growth, in order to maintain the equilibrium at the selected level.

Laboratory Equipment Design

In the laboratory investigation of pea-blancher wastes, the primary di-

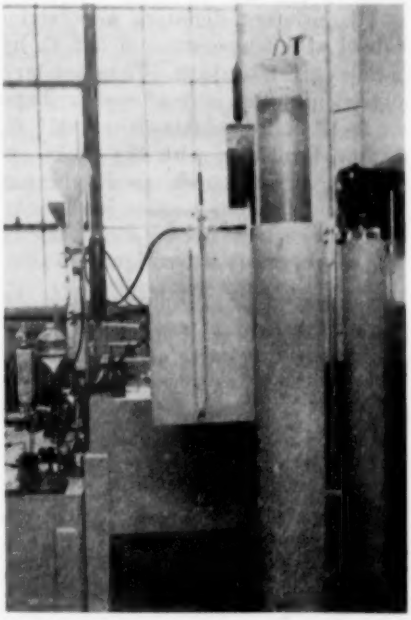

FIGURE 1.—Laboratory apparatus for digestion of pea-blancher wastes.

gesters used were made from 12-l. five-necked spherical flasks. The center opening or neck of each flask was provided with a mercury seal to accommodate the stirring device and to maintain anaerobic conditions. Two additional horizontal connections to the flasks reduced the available volume to approximately 10.7 l. Secondary digesters were constructed from 20-l. Pyrex bottles, in which mixing was provided by having the influent line extend to the bottom of the bottle.

The gas evolved was collected over a brine solution in a counter-weighted 6-in. diameter plastic tube. As illustrated in Figure 1, the brine solution was contained in a section of 8-in. diameter steel pipe about 5 ft. in height; the steel pipe was provided with a leveling base and brine level gauge. The inlet gas line was passed over a manometer filled with water, down the inside edge of the brine tank, across the bottom to the center and then up through the brine surface.

The primary digesters were maintained at a temperature of 37° C. by means of a water bath. The secondary digestion bottles were of such a shape as to be easily contained in a 37° C. hot-air incubator, with the necks extending through openings in the top cover.

Laboratory Investigations

The cycle time of 9 days used in the initial investigation was selected on the basis of prior research on the anaerobic fermentation of this pea blancher waste (30) (31). It was foreseen that the pH of the digester might fall. Sodium hydroxide was selected for the control of this condition. The use of sodium hydroxide did maintain the pH at the desired level (about pH 7) but the methane fermentation with the expected reduction in B.O.D. was inhibited.

The sodium hydroxide was suspected as the cause of unfavorable results. When the addition of sodium hydroxide was halted, the fermentation did not improve.

It was considered that a lower loading might stimulate digestion, and the cycle time was changed to 15 days. This did not benefit the primary digester, but the secondary digester began to show active signs of methane fermentation and B.O.D. reduction. Recirculation of secondary effluent, giving a cycle time of 10 days, was studied for a period of several months without improving primary digestion. During the latter part of this period additions of hydrogen sulfide, ferrous sulfate, calcium carbonate and calcium oxide were made in an attempt to stimulate digestion. Results were unfavorable.

Apparatus was assembled providing for two systems of primary and secondary digesters under nearly identical conditions. Three gas collectors were available. Two were attached to the primary digesters and one to a secondary digester. It was soon learned that over 95 per cent of the gasification took place in the primary digesters.

Operation of the digester systems on a ten-day cycle was started in operation on December 28, 1953. A ratio of two parts pea-blancher wastes to one part of sludge was employed. Both digesters were maintained at 37° C.

B.O.D. reductions were good. A previous study (1) indicates blancher waste to have a B.O.D. value of 35,000 p.p.m. If a conservative value of 30,000 p.p.m. is used, it is found that after four weeks of operation a reduction in B.O.D. of 94 per cent was taking place in the primary digester, and nearly 98 per cent in the complete system. Total solids content showed a reduction of about 79 and 89 per cent respectively. Volatile acids determinations made on the primary digester liquor during this period yielded an average value of 730 p.p.m. as acetic acid, the highest value being 910 p.p.m. A single determina-

tion on the secondary digester liquor indicated a value in the same range.

Studies of Higher Loading

A transition was next made from a 10 day cycle applying two parts pea-blancher waste and one part digested sewage sludge to an 8.3 day cycle with a feed of six parts blancher waste and one part sludge. The increased feed rate had no deleterious effect on the action of the digesters. The efficiency of B.O.D. reduction at the end of the investigation was 90 per cent in the primary and 94 per cent overall.

The next studies were based on a 10 day cycle applying two parts pea-blancher waste and one part recirculated secondary effluent. Again, B.O.D. reductions were satisfactory. On the basis of a B.O.D. of 30,000 p.p.m. for blancher waste, the reduction at the end of 28 days was 96 per cent in the primary stage and 98 per cent overall. Total solids at this date showed a reduction of 81 per cent in the primary and 91 per cent in the secondary.

Finally, a transition was made from a 10 day cycle of two parts blancher waste and one part recirculated secondary effluent to a 9 day cycle of blancher waste without recirculation.

Gas production averaged 724 cc. per hr. During the last three days of the period the average rate was 1,250 cc. per hr. or an increase of about 73 per cent. The increase in waste added from 712 ml. per day to 1,200 ml. per day is about 69 per cent. This is a close agreement and indicates that the greater loading together with the lack of recirculation had no deleterious effect on the digestion process.

The efficiency of the B.O.D. reduction based on the final data was 93 per cent in the primary stage and 95 per cent overall.

Foam appeared in the primary digester to such an extent after the transition, that it was necessary to install a trap in the gas discharge line to prevent passage of foam into the gas collector and manometer.

Assuming blancher waste to be simple sugar, the reaction shown below might represent the overall fermentation to methane:

$$2\ CH_2O \longrightarrow CH_4 + CO_2$$

It was determined that blancher waste contained approximately 31,000 p.p.m. or 3.1 per cent, volatile solids. During the 10 day cycle using a 2:1 sewage sludge ratio this would amount to an addition of 22.1 g. of solids per day. The theoretical production of methane from this amount of solids according to the assumed reaction would be about 8.3 l. per day.

The average rate of gas production at the end of 26 days was 701 cc. per hour, or 16.82 l. in one day. The average methane content during this period was 56.9 per cent, giving an actual methane production of about 9.6 l. per day. Applying a correction to 0° C. and 760 mm. of mercury this value is reduced to about 8 l. per day. This is a good agreement with the theoretical amount.

Applying the same reasoning to the parallel run using recirculation at the end of 28 days, a net daily methane production of 7.7 l. is indicated. This again is in good agreement with the theoretical production of 8.3 l. per day.

The loading, in terms of volatile matter contained in pea-blancher waste for the two trials, was 0.13 lb. per cubic foot per day.

Optimum rates of recirculation or seeding were not determined in this study. In the last phase of the study, however, recirculation was stopped and loading increased without any apparent impairment of efficiency. This loading on a 9 day cycle was 0.21 lb. per cubic foot per day.

Summary

The addition of sodium hydroxide to an anaerobic digester in the process of digesting pea-blancher waste was found

to be an undesirable method for the control of the reaction. After a prolonged period of sodium hydroxide additions the authors were unable to effect the recovery of the digester by decreasing the loading. A secondary digester in this same system gave reasonable results but recirculation from this secondary digester did not improve the primary digester.

A new investigation was started using two sets of duplicate digesters under nearly identical conditions. Both pairs of digesters were provided with equal amounts of blancher waste. One set received digested sewage sludge in an amount equal to one-half the blancher waste, while the other received the same amount of recirculated secondary effluent.

The volatile solids loading under these conditions equaled 0.13 lb. per cubic foot per day in the primary digester. The digestion cycle was 10 days. A reduction in B.O.D. of over 90 per cent was obtained in both cases.

The amount of blancher waste supplied to the digesters was increased to a 9 day cycle. One digester received an addition of digested sewage sludge in the ratio of one part of sludge to six parts of blancher waste, resulting in an actual 8.3 day cycle. The other received blancher waste without recirculation. The loading of volatile solids from the blancher waste under these conditions equaled 0.21 lb. per cubic foot per day. Digestion efficiency was not lowered. The pH value remained at about 7.5. Volatile acids did not rise. An increase in gas production directly porportional to the increase in loading was measured.

The success of the 9 day feed cycle served to corroborate the method of Adams and Hungate (30) for determining cycle times as applied by Mylroie, Maki and Hungate (31).

Acknowledgment

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ERRATA

10th Edition of Standard Methods for the Examination of Water, Sewage and Industrial Wastes

First Printing

p. 136, par. 3.3, 2nd line: change to read "0.31 g $Al_2(SO_4)_3 \cdot 18H_2$). . . ."

p. 252, right column: the last factor of the right side of the equation should be "12 I_2"

p. 253, par. 2.7, 2nd line: change to read "1.0*N:* Dissolve 248.2g. . . ."

p. 255, left column: in each equation, left side, numerator, place the values "P − u" in parentheses so that it appears "(P − u)x"

p. 264, par. 3.6, last word: should be "unreliable."

p. 308, right column: the present par. 3.10 *Ferrous sulfate* should be deleted and subsequent paragraphs renumbered to "3.10, 3.11 and 3.12."

p. 378, right column next to last line in the next to last par. preceding Section D: change the line to read "broth to a plate, thence to lactose."

First and Second Printing

p. 17, right-hand column, first full paragraph: change to read "As a general rule, results shall be expressed in terms of the substance actually determined. Exceptions to this, such as reporting boron as the element, B, rather than as borate, will be noted in the sections dealing with specific determinations."

p. 47, par. 2.3.1a: change the second line to read "and scale type sensitive to 0.25 micro-"

p. 126, par. 3.4: add a new sentence "Store in a cool, dark place."

p. 135, right-hand column: the chemical symbol in the numerator of the right hand of the equation should be "$Mg_2P_2O_7$."

p. 153, par. 1.1: change to read "The color system obeys Beer's law up to 0.18 mg/1 as nitrogen, or 0.6 mg/1 nitrite, with a 1 cm light path at 520 mu."

p. 170, right column. 3rd line: change to read "After 10 min, but before 15 min, using the same specific interval for all determinations, measure the color. . . ."

p. 197, top of right column: change last factor of numerator of fraction to read "411.5"

p. 339, par. 2.1: change third line to read "tilled water and dilute to 100 ml This"

WASTE WATER USE IN A SOAP AND EDIBLE-OIL PLANT *

By R. E. Sessler

Plant Engineer, Lever Brothers Company, Los Angeles, Calif.

The Lever Brothers soap and edible-oil plant located in the unincorporated area of eastern Los Angeles County is served by an extensive water conservation system which has been in successful operation for about four years.

In the past it has been rather common practice for the company to select plant sites on the shores of lakes or rivers from which large quantities of cooling water could be passed through the plants with no problem of water supply or waste water disposal. For the Los Angeles Plant, however, it became necessary to provide a water system quite different from those in general use in its eastern plants.

Lever Brothers Co. is engaged in the manufacture of soaps, synthetic detergents and vegetable oil shortening, and each pound of production requires about 10 gal. of water, most of which is merely circulated and not consumed. The original plans for this plant were based on a water requirement of 6 m.g.d. with a future requirement for 12 m.g.d.

Planning

The consulting engineers started water requirement studies in 1948 to develop a plant water service. In a review of the available water sources, it was determined that the California Water Supply Company, which serves the area of the plant site, could furnish water to the plant at a maximum rate of 2,000 g.p.m. at a cost of $200.00 per million gallons. Since the plant would require as much as 7,000 g.p.m. at some peak periods, it was obvious that the local supply would be inadequate for a single-pass system. The high cost of the water made the local supply even less satisfactory as a means of serving the full plant requirements.

A thorough investigation was conducted regarding the economy and advisability of drilling private water wells at the plant site. There appeared to be some economic advantage to the use of private wells. However, the idea was abandoned due to the uncertainty of water controls as evidenced by the involved history of litigation over the allocation of underground water rights. In addition, there is the added uncertainty of the underground water supply.

In addition to the problems of water supply, some consideration had to be given to the available facilities for disposal of large quantities of water. In February, 1949, it was disclosed by the Los Angeles County authorities that the discharge of sanitary and process wastes from the plant into the sanitation district sewer system would be limited to an unofficial maximum flow rate of 2 m.g.d. This, together with the strict prohibition of contaminated waters entering the county storm drain system, presented a problem requiring complete analysis of all expected wastes, and the development of a practical and economical method of disposal.

Samples of plant waste water were taken at the Hammond, Indiana plant of Lever Brothers Co. and analyses made to determine the character and

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

acceptability for discharge to the Los Angeles County waste systems. From these analyses it became evident that to discharge any plant wastes, including the cooling water, into the county storm drain system would not be a satisfactory method of disposal.

In further discussions with the authorities, it was decided that a "continuous flow allocation" of untreated sanitary sewage could be discharged to the county sanitary sewer. Wastes in excess of this allocation would be treated to meet the standards required for discharge to the county storm sewer.

It became quite obvious at this point that the treated waste water would be clean enough to re-use. The greatest quantities of water to be discharged were those used for cooling purposes, and these had relatively little contamination to be removed. Based on these points, and the high cost of water, it was decided that an extensive system should be developed to recirculate cooling water throughout the plant. This system should include a cooling tower and equiment to remove contaminants from the water.

Water Utilization System

The general water system for the plant was arranged as follows:

1. The water supply was to be furnished by the California Water Service Company.
2. A closed system, utilizing an induced draft cooling tower, would circulate 8,000 g.p.m. of cooling water.
3. The discharge of normal quantities of rain water from roofs and roadways would be directed into the Los Angeles County storm sewer system, but in case of exceptionally heavy rainfall, which would cause flooding, an emergency valve arrangement would allow outdoor waste water sumps to discharge into the storm sewer system. During normal operations there would be no flow of waste water into the storm drains.
4. Industrial waste water too contaminated to be economically cleaned for re-use would be directed to the sanitary sewer.
5. All wastes from toilets, wash basins and other plumbing fixtures would be discharged directly into the sanitary sewer.

The water circulation system was developed into a well coordinated arrangement. The large volume of water to be circulated was divided into two classes: (a) clean, uncontaminated water used in cooling jackets, coils, tubular condensers and chill rolls, or wherever the water does not come in direct contact with any product, and contamination is not generally expected, and (b) water used in barometric condensers and water that comes in contact with oils or fats so that it is normally contaminated to some extent. For this second class of water it was necessary to provide some means for removal of the contaminants in the return line to the cooling tower. Since the plant is divided into two sections, the Detergent Division and the Edible Oil Division, the classes of water were further divided into classes of clean and dirty water from each division.

To serve these requirements three separate water circulation systems were established: (a) Edible Oil Division dirty, (b) Detergent Division dirty, and (c) Combined clean water from both divisions. The dirty water systems were kept separate in order to allow independent recovery of the oils or fats which might be present. Combining the clean systems was satisfactory because contamination was expected to be slight.

The three systems were circulated through a single, 8-cell cooling tower. Within the tower there are concrete dividing walls in the hot and cold basins to separate these three systems. Make-up water is fed to the hot basin

of the clean system, and make-up to the two dirty sections is delivered by overflow from the clean section. Overflow from the lowest dirty section is to the sanitary sewer.

Air Flotation Treatment

For the two dirty systems it was necessary to provide equipment for the removal of suspended solids in the form of free fat and oil. The air flotation method of handling waste waters of this type was recommended as a method which would produce a satisfactory waste water effluent. Typical samples of waste water from a similar midwestern plant were submitted for small-scale performance tests by air flotation. The results were encouraging, and the method was considered adequate.

For materials which do not respond sufficiently to air flotation alone, chemicals are introduced to provide flocculation. The addition of chemicals has been avoided because they would render the recovered oil and fat unsuitable for use, and satisfactory performance has been obtained without them.

For the Detergent Division dirty water system a 1,500 g.p.m. air flotation unit was installed, and a 2,000 g.p.m. unit was provided for the Edible Oil Division dirty water system. Both were located adjacent to the cooling tower (Figure 1). An additional unit was installed in the Soap Division and one smaller unit each in the soap and edible oil processing departments.

The Edible Oil Division unit operates normally on a flow of about 1,400 g.p.m. of waste which is quite heavy in fatty matter. In some of the vacuum vessels in the Edible Oil Processing Department, the oil is subjected to live steam stripping. This results in the entrainment of many particles of fatty matter in the vapors which are pulled

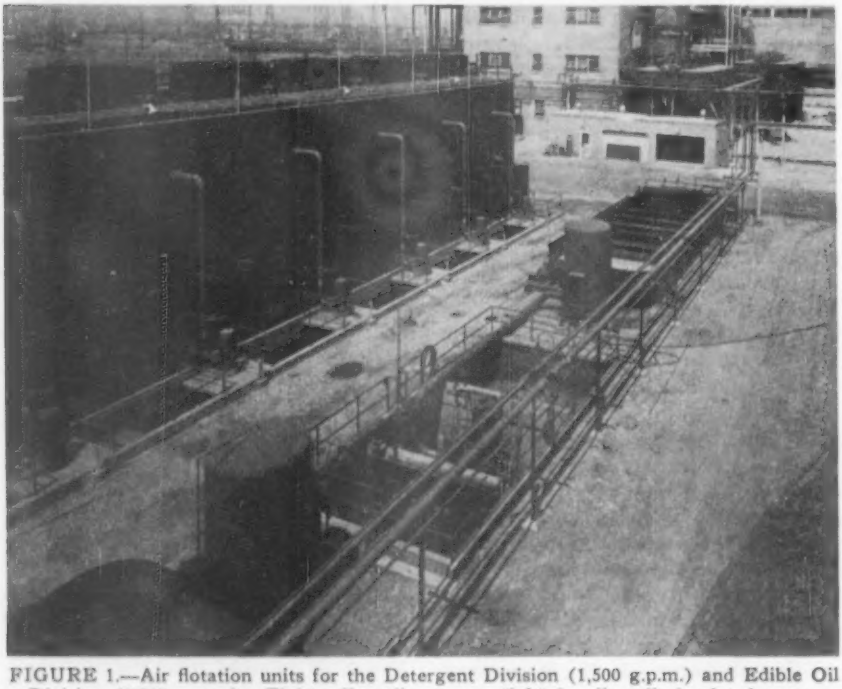

FIGURE 1.—Air flotation units for the Detergent Division (1,500 g.p.m.) and Edible Oil Division (2,000 g.p.m.). Eight-cell cooling tower (left) handles all circulated water.

from the vessel and condensed in the cooling water. The fatty matter seems to enter the cooling water in the form of rather large suspended particles which respond well to the air flotation method of separation. Average operating results follow:

Item	Influent	Effluent	Per Cent Removal
Total solids (p.p.m.)	6,000	1,200	81
Suspended solids (p.p.m.)	5,000	200	96
Dissolved solids (p.p.m.)	1,000	1,000	—
Total fatty matter (p.p.m.)	3,900	40	99

The air flotation process removes approximately 96 per cent of the suspended solids, and practically all the fatty matter.

The flotation unit in the Soap Division normally operates on a flow of 800 g.p.m. of waste water which is practically free of oil and grease. There is little vacuum treatment of the oil and tallow in the soap processing, and there is no steam stripping. The water does accumulate some suspended solids, but the concentration does not build up to more than 1,000 p.p.m.

In the cooling tower water there is a build-up of dissolved solids as a result of evaporation. This is held below 2,400 p.p.m. by blow-down. The fatty matter and B.O.D. in the blow-down are just high enough to make it advisable to discharge the blow-down to the sanitary sewer rather than the storm sewer.

The cooling tower water is conditioned with phosphates to reduce scale formation and with bromine to kill the algae. A fairly heavy quantity of sludge settles to the bottom of the inlet and outlet water sumps of the cooling tower. This consists primarily of dirt and dead algae, but it is relatively free of fatty matter. Once each year the sumps are drained and the sludge is removed to a dump. The pH of the water is maintained between 7 and 8. No pipeline clogging or interference of any type has been experienced with the free circulation of the cooling tower water throughout the plant. The cost of the cooling tower, the air flotation units and the water circulation system is saved every three years by reduced water costs.

From the soap and edible oil processing departments there is a certain amount of rather heavily contaminated waste water for disposal. This consists of floor washings, tank washings, water settled out of oil, neutralized acid oils, etc. By skimming this water in retention basins and then passing it through air flotation units a sufficient amount of the contamination is removed to render the water suitable for discharge into the Los Angeles County sanitary sewer lines. The small flotation unit (125 g.p.m.) in the Soap Oil Process Department and the similar one in the Edible Oil Processing Department handle only about 6,000 gal. and 30,000 gal. of waste water per week, respectively.

The solids and fatty matter in this water total approximately 10,000 p.p.m. as it passes to the air flotation unit after the skimming operation. The fatty matter is more finely dispersed in an emulsified state, which greatly reduces the effectiveness of separation by flotation. The waste water discharged contains about 1,000 p.p.m. of fatty matter which is practically all finely dispersed. Normally there is about 20 p.p.m. of floatable oil.

Summary

The Los Angeles County Sanitation District policy governing district trunk sewers states that industries with wastes containing animal or vegetable oils or fats, mixed with other suspended matter rendering separation difficult, may in some cases be allowed concentrations of floatable oil or grease up to 25 p.p.m. and dispersed oil or grease in

concentrations up to 600 p.p.m. provided this matter does not settle out of solution.

The small volume of waste water from the air flotation units is mixed with the heavy flow in the sanitary sewer lines within the plant. For the combined flow the total fatty matter is under 200 p.p.m. when it enters the county sanitary sewer lines. There is little floatable oil in the waste water, and there is no settling out of the dispersed fatty matter. The B.O.D. has never been considered critical and occasional tests have substantiated this.

Industrial waste water from a soap and edible oil manufacturing plant can be successfully processed for reutilization and disposal by flotation separation methods. At the same time considerable economies can be had in the conservation of water.

NON-MEMBER MEETINGS OF INTEREST

The Inter-American Association of Sanitary Engineering will hold the Fifth Inter-American Congress of Sanitary Engineering at the Engineering School, University of Peru, Lima, Peru, March 18–24, 1956. Details of the meeting program, accommodation arrangements and exhibits may be obtained through Thomas P. Browne, Export Manager, The Permutit Company, 330 West 42nd St., New York 36, N. Y.

A hydraulics conference will be sponsored by the Kansas City Section, American Society of Civil Engineers at the Continental Hotel, Kansas City, Mo., November 21–22, 1955. Details are available from Glenn C. Gray, Chairman, Sub-Committee on Publicity, Hydraulics Conference, 19 East Gregory Blvd., Kansas City 14, Mo.

The 48th Annual Meeting of the American Institute of Chemical Engineers will he held November 27–30, 1955 in Detroit, Mich. Details can be obtained from AICHE headquarters, 25 West 45th St., New York 36, N. Y.

Stream Pollution

SUGGESTED CLASSIFICATION OF ALGAE AND PROTOZOA IN SANITARY SCIENCE

By C. Mervin Palmer and William Marcus Ingram

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Many types of microorganisms are of real importance in the field of sanitary science. Bacteria, molds, yeasts, protozoa and algae all play significant roles in relation to water and sewage. A limited number of them are pathogenic but the great majority are those that cause nuisance conditions in water supplies or are associated with sewage treatment and stream self-purification.

Classification of Pigmented Flagellates

Confusion exists in the literature of sanitary science in the classification of a considerable number of microorganisms. The confusion is especially evident in dealing with certain organisms which are on the borderline separating algae of the plant kingdom from protozoa of the animal kingdom, where one investigator may classify a particular organism with the algae and another place the same organism with the protozoa (1) (2).

The organisms most often involved in this confusion are those known as pigmented flagellates which have both the protozoan characteristic of being able to swim by means of flagella, and the algal characteristic of photosynthesis made possible by the presence of the pigment chlorophyll. Thus, they are intermediate between typical algae and typical protozoa, and it would depend upon which characteristic was emphasized as to whether they would be listed in the plant kingdom as swimming, flagellate algae, or in the animal kingdom as photosynthetic pigmented * protozoa. It is the authorities in the fields of protozoology and algology who are responsible for the existing confusion, since they have come to no agreement as to the classification of the organisms involved.

In an attempt to resolve opinions among authorities concerned with the characteristics an organism should have in order to be classified as a protozoan or as an alga, a large group name, the Protista, was proposed by Haeckel (3). Under it all protozoa and all one-celled algae were lumped together without distinction. The term "Protista" has not received general recognition and would be of little or no value to the sanitary scientist.

Existing Confusion in Sanitary Science

A recent book, "Water Quality Criteria" (4) serves to illustrate the existing confusion in classification of microorganisms. The pigmented flagellate, *Synura* is listed on page 170 as an alga, and later, on page 333, as a protozoan. Lackey (5) in his paper, "Protozoan Plankton as Indicators of Pollution in a Flowing Stream," referred many or-

* "Pigmented" refers to chlorophyll and other photosynthetic pigments only.

ganisms to the protozoa that he, in other papers, classified as algae, (1) (6). Turre (7), who has published photomicrographs of algae in water supplies, includes *Volvox* with the green algae, while *Dinobryon* and *Synura* are listed under protozoa. He considers the protozoa as one group of algae.

Mohr (8), in his paper on protozoa as indicators of pollution, lists the chlorophyll-bearing *Euglena* with the organisms *Paramecium* and *Vorticella*, which are nonpigmented protozoa. Brinley (9) lists the pigment-containing *Ceratium* and *Peridinium* as protozoa. Thomas and Grainger (2), in their book, "Bacteria," refer the pigmented flagellates *Chlamydomonas, Uroglena, Synura,* and *Dinobryon,* to a class of protozoa, the Mastigophora. Cox (10), in his book, "Laboratory Control of Water Purification," lists the following as protozoa: *Dinobryon, Euglena, Uroglena, Synura,* and the unrelated nonpigmented protozoan parasite *Endamoeba histolytica.* Hale (11), in recommending chemicals required for control of organisms, lists ten genera, including *Chlamydomonas,* under protozoa with only two nonpigmented genera in the list, namely, *Bursaria* and *Endamoeba.* Other pigmented flagellates, including *Pandorina* and *Volvox* which are closely related to *Chlamydomonas,* are placed with the algae. Hopkins (12) lists ten genera of chlorophyll-bearing organisms under protozoa in a table reproduced from Hale (13). Whipple, Fair, and Whipple (14) list some of the pigmented flagellates as protozoa and others as algae. In "Water Quality Criteria" (4), *Synura, Dinobryon,* and *Uroglenopsis,* all containing photosynthetic pigments, are listed under protozoa in a discussion of domestic water supplies along with such true nonpigmented protozoan parasites as *Endamoeba histolytica* and *Balantidium coli.* Pigmented forms such as *Gymnodinium, Gonyaulax* and *Peridinium* are listed as protozoa in a discussion of their relation to fish and shellfish mortality. *Gonyaulax catenella,* a pigmented form responsible for mussel poisoning, is considered only as a protozoan.

Writers in the field of sanitary science who have placed pigmented flagellates under the algae include Kehr *et al.* (15), Hobbs (16), Prescott (17), Brinley and Katzin (18), Gainey and Lord (19), and Sorensen (20). Modern workers in the field of algology also follow this practice, including Fritsch (21), Smith (22), Tiffany and Britton (23), and Prescott (24).

Significance of Oxygen-Producers

The relationship of microorganisms, including the flagellates, to oxygen is particularly significant, especially when considering their role in treatment of sewage and in stream self-purification. The amount of dissolved oxygen is one of the limiting factors in determining the speed with which microorganisms will bring about the modification of sewage in a treatment plant or of organic wastes in a stream.

It is assumed from results of research, to date, that all organisms containing chlorophyll and other related pigments are capable of carrying on the process of photosynthesis. In this process the organisms remove carbon dioxide and water from the environment and, in the presence of light, produce oxygen and carbohydrates. Much of the oxygen is released by the organism into the water, whereas the carbohydrate is retained. The photosynthetic organisms are therefore recognized as oxygen producers.

Nonphotosynthetic organisms undergo no process by which oxygen would be released into the water. These nonpigmented forms carry on the process of respiration in which the relationship of carbon dioxide and oxygen are the opposite to that in photosynthesis since available oxygen from the environment is utilized and carbon dioxide is released. The pigmented organisms also carry on respiration in addition

to photosynthesis, but during the hours of daylight the latter takes place at a much higher rate. The supersaturation of natural waters with oxygen, which is frequently encountered, is usually a result of photosynthesis. In darkness, with no photosynthesis taking place, the pigmented organisms behave in a manner similar to the nonpigmented, both groups using oxygen and releasing carbon dioxide. When the sum of the effects occurring in a typical diurnal-nocturnal cycle are considered, however, the pigmented organisms are recognized as oxygen-producers and the nonpigmented ones as oxygen-consumers. If present and active in large numbers, the oxygen-producers stimulate oxidation of organic wastes by the oxygen-consuming organisms. Both oxygen-producers and oxygen-consumers, therefore, are distinct and important groups of microorganisms for the sanitary scientist to consider.

This relationship of microorganisms to oxygen is also utilized as one of the basic characteristics in their classification. *Typical* algae are described as oxygen-producers, and the *typical* molds, protozoa, yeasts, and bacteria as nonoxygen-producers or oxygen consumers. The confusion in the classification occurs when this characteristic is not given sufficient emphasis.

A Solution to the Problem

When the oxygen and non-oxygen producers are mixed together in a classification, it is not possible to evaluate clearly their roles in water pollution problems. As has already been indicated, this type of confusion is evident in connection with the classification of the pigmented flagellates. For the sanitary scientist, it would not be difficult to overcome the confusion, because to him the oxygen-production of these organisms needs to be taken into consideration, whereas their swimming ability is of little or no importance. The sanitary scientist, therefore, should logically group the pigmented flagellates with the photosynthetic algae rather than with the nonpigmented protozoa.

In actual practice it might be difficult or impossible to recognize oxygen-production by individual organisms were it not for the fact that the visible characteristic of pigmentation is associated with that physiological phenomenon. These organisms normally possess the green chlorophyll and frequently additional pigments in amounts sufficient to be visible under a compound microscope. The pigments are located within the protoplasm of the individual cells and can be seen to be localized within the cells of the organisms in the form of one or more plastids or chromatophores. The other pigments which may be present in addition to the chlorophyll lend various shadings of color to the plastids in the cells. The shades most frequently encountered in the pigmented flagellates are green, yellow-green, and brown. Examples of some of the pigmented flagellates with their plastids or chromatophores are illustrated in Figure 1, together with some non-pigmented flagellates. It will be noted that, in either group, both unicellular and colonial types are to found.

In addition to the sanitary scientist's need to separate organisms according to their oxygen relationship, it is desirable from another standpoint to have all organisms grouped according to one recognized classification in order to eliminate the wasteful duplication of effort. For example, if the same swimming, pigmented organism is causing an undesirable odor in a settling basin at a water treatment installation in Iowa and at another in South Dakota, it is a handicap to have the organism listed as an alga at one place and as a protozoan at the other. Because of the presence of the photosynthetic pigment, it is recommended that all workers list it as an alga. Coming to such an agreement would do away with doubt that may arise in comparing effective

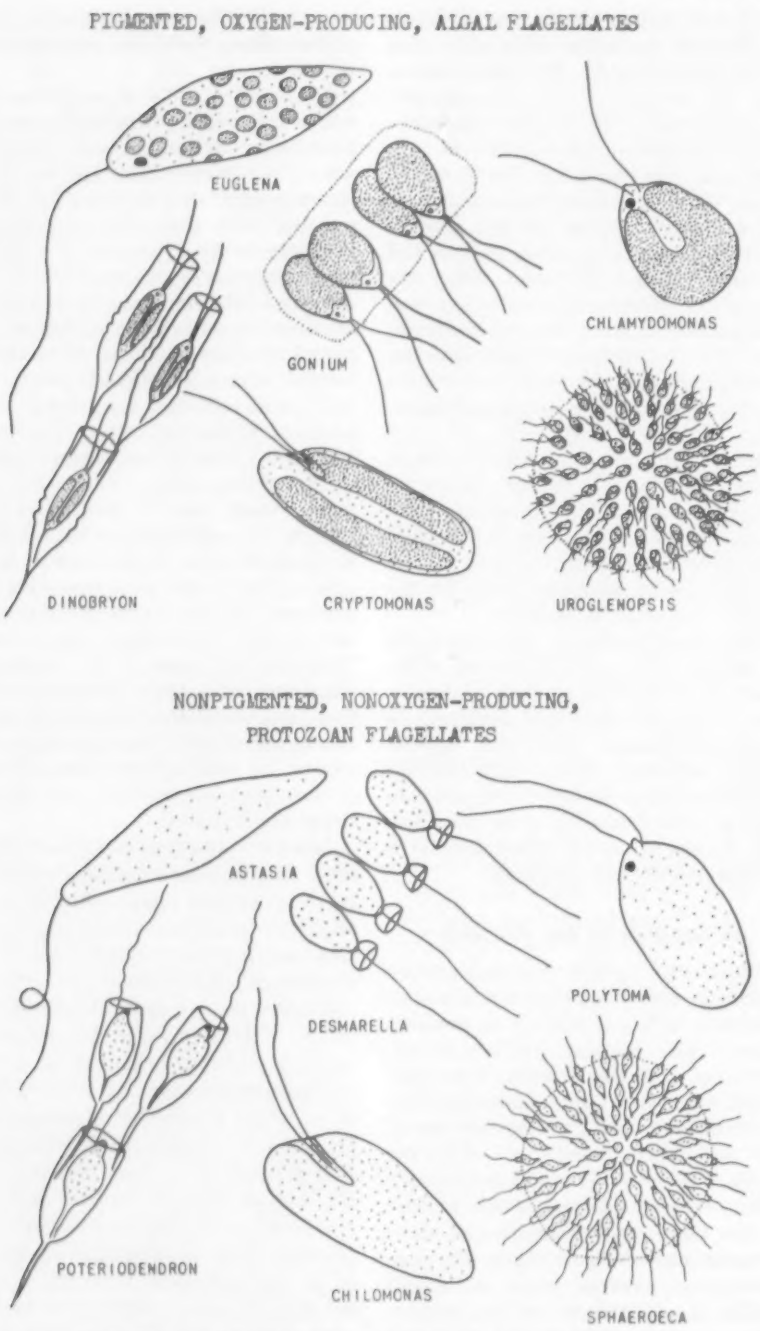

FIGURE 1.—Typical pigmented and nonpigmented flagellates.

TABLE I.—Classification of Flagellates

Pigmented, oxygen-producing "algal" flagellates		Nonpigmented, nonoxygen-producing "protozoan" flagellates	
Carteria	Hemidinium	Anisonema	Mastigamoeba
Ceratium	Lepocinclis	Anthophysis	Mastigella
Chlamydomonas	Mallomonas	Astasia	Menoidium
Chlorogonium	Ochromonas	Bicosoeca	Monas
Chromulina	Pandorina	Bodo	Monosiga
Chroomonas	Peridinium	Cercobodo	Noctiluca
Chrysococcus	Phacotus	Cercomonas	Notosolenus
Chrysosphaerella	Phacus	Chilomonas	Oikomonas
Colacium	Pleodorina	Clautriavia	Peranema
Cryptoglena	Pteromonas	Codonosiga	Petalomonas
Cryptomonas	Pyramimonas	Cyathomonas	Pleuromonas
Dinobryon	Pyrobotrys	Desmarella	Polytoma
Dunaliella	Rhodomonas	Dinema	Polytomella
Eudorina	Spondylomorum	Dinomonas	Poteriodendron
Euglena	Synura	Distigma	Rhabdomonas
Eutreptia	Trachelomonas	Entosiphon	Sphaeroeca
Glenodinium	Uroglena	Euglenopsis	Tetramitus
Gonium	Uroglenopsis	Heteronema	Trepomonas
Gonyaulax	Volvox	Hexamita	Tropidoscyphus
Gymnodinium	Wislouchiella	Hyalogonium	Urceolus
Haematococcus		Khawkinea	

control measures put into practice in each area.

Nonpigmented Flagellates

A second small group of flagellates is involved in the confusion over classification. A few nonpigmented, swimming organisms are considered by authorities to be so closely related to the pigmented swimming forms that they are often placed in the same groups with the latter rather than with other nonpigmented forms. Included among the nonpigmented flagellates, which have been involved in the confusion, are such genera as *Astasia, Polytoma, Chilomonas, Euglenopsis,* and *Peranema.* Thus, an algologist would consider these particular nonphotosynthetic organisms to be algae, just as many protozoologists have considered the pigmented flagellates to be protozoa.

Again, the sanitary scientist can settle this problem for himself by placing all nonpigmented (nonoxygen-producing) flagellates with the protozoa, disregarding the close evolutionary relationship between certain pigmented and nonpigmented forms.

Recommended Grouping of Flagellate Forms

Table I which follows, separating flagellates into two easily recognizable groups, can serve as a guide to classification for those working in sanitary science, and thus, make it possible to overcome the existing confusion. The first group includes the pigmented, photosynthetic, oxygen-producing, "algal" flagellates. The second group includes the nonpigmented, nonphotosynthetic, nonoxygen-producing, "protozoan" flagellates. In a very few cases, certain species of the genera in the first list are nonpigmented. These particular species would, of course, need to be placed in the second list.

More than 150 genera of flagellates have been reported for the United States. Some of these forms are comparatively rare and are of little or no importance to sanitary scientists. Others, however, are included among the microorganisms frequently encountered in water supplies or sewage. Organisms listed are limited to those flagellates which are considered to be of significance to the sanitary scientist.

Summary

A lack of agreement exists among botanists and zoologists as to a definite line of demarcation between algae and protozoa. Consequently, there is no uniformity in the classification of a considerable number of flagellates which are important in the field of sanitary science. It is recommended, therefore, that the presence or absence of photosynthetic pigments (indicating the ability or inability to produce oxygen) be used in this field of applied science to separate the flagellates into the pigmented (algal) and nonpigmented (protozoan) types.

Acknowledgments

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BURSTING BUBBLES AND AIR POLLUTION *

By A. H. Woodcock

Woods Hole Oceanographic Institution, Woods Hole, Mass.

Photographic studies of small bubbles bursting at the surface of fresh and sea water have shown that aerosols are produced through the breakup of minute water jets formed by the collapse of the bubble cavities (1). It is the purpose of this paper to show a few of these photographs as evidence of the nature of the bubble-jet-droplet mechanism, and to suggest that this mechanism may cause bacteria to be ejected into the atmosphere from natural water surfaces. The interest of the author has centered around a study of the meteorological role of bubble-produced aerosols. However, studies of bursting bubbles as sources of natural aerosols have left a strong impression that these bubbles and the weather conditions which produce them may be of considerable significance in bacteriology.

Numerous studies have demonstrated the role of aerosols in the distribution of infectious and noninfectious bacteria (2) (3), and in the transport of bacteria by aerosols resulting from bubbling or splashing procedures in the laboratory (4). However, none of these studies of bubbles and air-borne droplets has indicated the nature of the mechanism of aerosol production by bursting bubbles, nor the fact that bubbling and aerosol production in natural waters is a common occurrence associated with a variety of weather conditions. Accordingly, the author's observations concerning bubbles and droplets may be useful to those working on the problems of the origin of air-borne infection and pollution.

* Contribution No. 761 of the Woods Hole Oceanographic Institution.

Bubble-Jet-Droplet Mechanism

Figure 1 shows photographs of small bubbles forming jets and droplets. Similar photographs have been made of bursting bubbles 200 μ in diameter. Upon bursting at the surface of sea water, bubbles of a size range of about 20 to 2,000 μ in diameter are observed to produce droplets of about 2 to 200 μ in diameter, the larger bubbles forming the larger droplets. Each bubble produces about five droplets which are projected into the air 0.1 to 15 cm. above the surface (Figure 2), the larger bubbles projecting droplets higher. The smaller "microscopic" bubbles produce, upon bursting, much smaller droplets than those pictured. The average diameters of the droplets produced when a bubble bursts are usually about one-tenth of the diameter of the bubble.

At relative humidities commonly found over the sea, droplets of sea water of this size will evaporate quickly to form nuclei of about 1 to 45 μ in diameter. Such nuclei fall slowly (0.01 to 13 cm. per sec.), and atmospheric turbulence and winds cause them to be carried many miles and hundreds of feet into the air (5). Nuclei from droplets produced at the surface of less saline waters (lakes, rivers, etc.) would of course be smaller and hence have a lower settling rate.

Nature of Droplets From Jets

Measurements in the laboratory have shown that the bubble-jet-droplet mechanism is effective in removing films from water surfaces. This is readily demonstrated by placing an oleic acid

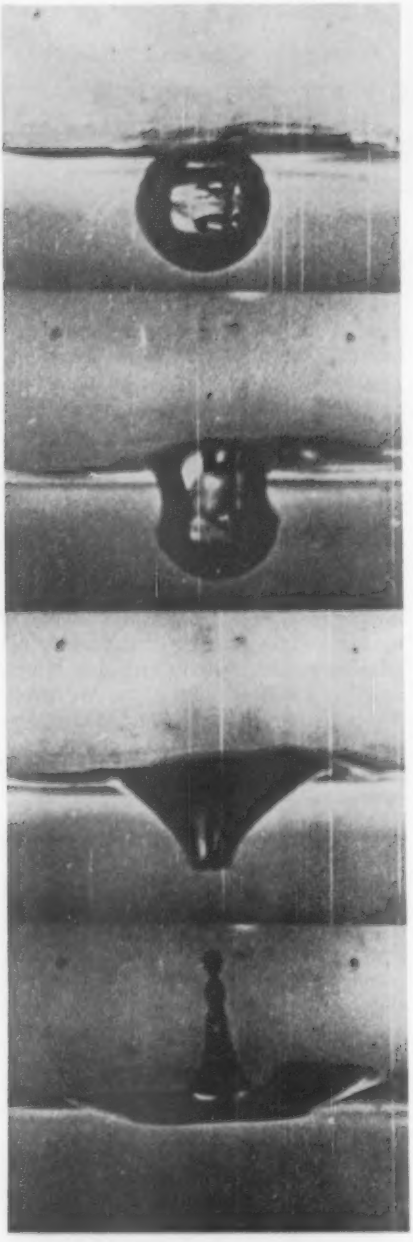

FIGURE 1.—Stages in bubble collapse and jet formation. Bubble diameter 1.7 mm.; time interval between top and bottom pictures 0.002 second.

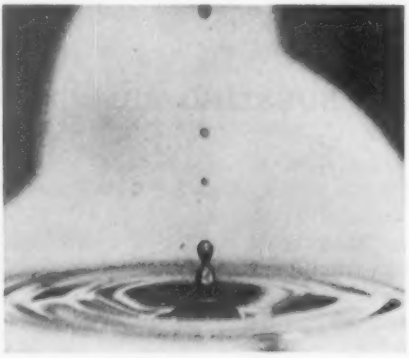

FIGURE 2.—Oblique view of jet and droplets from 1 mm. diameter bubble. Diameter smallest droplet 90 μ.

film on a water surface under which small bubbles are introduced through a fine capillary tube. Droplets produced by these bubbles, when allowed to fall upon a clean water surface which has been dusted with lycopodium, cause a characteristic oleic acid film to displace the powder in circular patches. Similar droplets, when produced at an uncontaminated water surface, show no displacement of the lycopodium dusted surface. This simple experiment shows that the mechanism can remove liquid from exceptionally thin surface layers. In fact, the mechanism is useful in completely removing contaminating films from water surfaces. Thus, aerosols produced by small bubbles may contain materials which are not present in the bulk of the water mass from which the aerosol is produced. A natural occurrence of this film-removing mechanism is indicated later in discussing the "red water" phenomenon of the Gulf of Mexico.

Since the bubble-jet-droplet mechanism removes materials from the superficial film of the water in the laboratory, it probably removes organic films present on natural water surfaces (6) as well. When bubbles are present in these waters, the nature of this film and the number of organisms in it be-

comes important to those concerned with air pollution.

Observations of Natural Aerosols

There is increasing evidence that droplets, arising naturally from sea water surfaces, sometimes contain materials other than water and sea salts. Zobell (3) found viable marine bacteria deposited on culture media exposed near the sea, and suggested that these marine bacteria were carried by seawater droplets arising from the breaking of waves. He showed that some of these air-borne bacteria survive despite drying and exposure to sunlight.

During the "red water"* plankton bloom in the Gulf of Mexico, the air over the discolored sea water became quite irritating to the human respiratory tract. Woodcock (8) showed this irritation occurred only when bubbles were present on the sea surface, and he was able to reproduce the effect in the laboratory by inhaling aerosols produced by small bubbles bursting at the surface of stored "red water."

Microscopic examination of the aerosols from "red water" revealed that their surface tension was much reduced, compared to the surface tension of droplets of relatively clear sea water, and that some of the droplets contained parts of plankton organisms (9). It was clear that the droplets from the bursting bubbles were acting as a mechanism for the removal of an irritating agent from the water and for the dispersion of this agent in the air.† The bulk "red water" was, however, entirely non-irritating.

Evidence that the bubble-jet mechanism may produce droplets containing organic materials from the surface film of natural waters is reason to consider whether or not these droplets might contain pathogens. Zobell (3) and Proctor (12) found no pathogens among cultures of a great many aerosol samples taken at various altitudes and locations. However, there is no indication that any of the samples were taken at a place and time at which such organisms might have been most concentrated. For instance, no one seems to have made a study of the bacterial population of aerosols taken, with a modern impact sampler (13), directly downwind from a water area contaminated by sewage, though these areas are known to contain pathogens (14). Would aerosol samples taken in this manner contain pathogens?

A study which attempts to answer this question should be extended over a sufficient period of time to obtain results representing a variety of weather conditions which may effect the number of air-borne bacteria. Some of these important factors are the amount of sunlight, the relative humidity, and the amount of mixing within the air itself (15) (16) (17). The presence or absence of conditions which favor the concentration of bacteria in the surface film and cause the production of great numbers of small bubbles in the surface waters are significant.

Bubbles in Natural Waters

It is the presence of bubbles which is emphasized here, for it is this condition that causes a rapid transport of surface film waters into the atmosphere

* Sea water which was toxic to fish and which contained great numbers of the dinoflagellate of the genus *Gymnodinium*, see reference (7).

† It was suggested that the substance causing the respiratory irritation and, presumably, also killing the fish might be present in the sea and in the droplets as a surface-active organic material. It was also suggested that samples of this material might be obtained for analysis by filtering the aerosols arising from (experimentally introduced) bubbles bursting at the surface of "red waters." This mode of attack upon the problem of isolating the "red water" poison was not pointed out in the summarizing studies by Galtsoff (10) and by Brongersma-Sanders (11), though both authors concluded from a study of the literature that a toxic substance, produced in living plankton, is probably the cause of the mortality.

as droplets. Great numbers of small bubbles are produced in natural waters by the impact of rain drops, by the trapped air among the ice crystals of sleet, hail and snow and by the "white caps" caused by winds. For example, large snow flakes falling into and melting in sea water are observed to introduce microscopic bubbles (10 to 100 μ in diameter) at a rate of about 25 per sq. cm. of water per second. If these bubbles produce, upon bursting at the surface of natural waters, numbers of droplets comparable to similar bubbles in the laboratory, droplets will be ejected into the overlying air at a rate of about 100 per sq. cm. per second. Many of the droplets so produced are too small to fall out of the air quickly, or to be readily washed out through impingement upon precipitation particles.

The author hopes that these remarks will aid bacteriologists in the investigation of the role of bursting bubbles in natural waters as sources of air-borne pathogens.

Acknowledgment

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SUGGESTED METHOD FOR REPORTING BIOTIC DATA

WILLIAM M. BECK, JR.

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The use of biological methods in stream condition evaluation in recent years has been increasing constantly. One phase of this activity badly neglected by biologists is the problem of reporting the interpretation of biological results to individuals trained in other fields. Reports from biologists to engineers, chemists, and other technical personnel fall into one of two categories. These reports are either excessively wordy dissertations on biological principals, physiology, life-history or classification of organisms, or they are phrased in the terminology of a primer. Neither form is effective for the former is discouraging and the latter insulting.

It is the purpose of this report to present methods of handling biotic data in which these problems may be minimized if not eliminated entirely. These methods have been in use in Florida for a period sufficient for thorough testing under a great variety of conditions and have proved satisfactory.

The technical aspects of aquatic biology are the problems of the biologist, not of the engineer directing his work who is interested in the interpretation of the biotic data. The bacteriologist reports results in terms of most probable number and the chemist uses parts per million and a standard pH scale. The biologist, however, has been without a scale for use in reporting results or interpretations.

The biological background for the present report has been published elsewhere (1). However, a brief review of the classification of organisms used in this method is pertinent.

The methods used in Florida are based on the classification of selected macroscopic invertebrates (aquatic insects, crustaceans, molluscs and worms) with regard to organic pollution. Class I organisms have been found to tolerate no appreciable organic pollution. Class II organisms will tolerate moderate organic pollution, but cannot exist under conditions approaching or reaching the anaerobic. The other classes need not be considered here.

Any generalities made in this report apply only to the macroscopic invertebrates existing in Florida. It should be pointed out that these methods are confined to fresh waters and encroaching salinity has a marked effect on the fauna of a stream.

The Biotic Index

Shortly after the classification of organisms was developed, it became apparent that the inability to present biological data in graphic form was a major problem. In stream quality surveys, values representing dissolved oxygen, biochemical oxygen demand, alkalinity, pH, most probable number etc., could be presented quickly and effectively by graph or chart, while biological results remained discussional.

On first consideration, it appeared that a single value representing faunal evaluation should be based on a combination of the number of species of clean water organisms (Class I) and the total number of species comprising the fauna of that section of the stream in question. There were two reasons why this initial attempt was aban-

* Presented at 1955 Annual Meeting, Florida Sewage and Industrial Wastes Assn.; St. Petersburg, Fla.; Nov. 8–10, 1954.

doned. While a large number of species (a diverse fauna) is typical of clean water, it is even more typical of diverse living conditions. Therefore, it was not an entirely reliable consideration. The second reason was this initial calculation did not take into consideration the presence of Class II organisms which, though tolerant of a moderate amount of organic pollution, do reveal something with regard to water quality. Apparently the measure should be based on the numbers of species comprising Classes I and II and should not include the total number of species comprising the fauna.

The simplest value based on the above considerations should be a total of the numbers of species in the first two classes. The resulting value was tested on over 300 collections from stations or streams of known conditions. One weakness immediately evident in the interpretation of this index was that the higher the index, the greater the indication of healthy stream conditions. There was a major area of overlap in the case of low index values for certain types of clean streams and relatively high indices for some moderately polluted streams. Simple arithmetic calculation revealed that this overlap could be minimized, but not eliminated.

One final consideration remained. Should the index be based on statistical or arithmetic methods? In order to keep the index as simple as possible the arithmetic method was chosen.

The final method for the calculation of the index was selected by reasoning. As mentioned previously, the zone of overlap between the index for a clean stream and the index for a moderately polluted stream could be reduced but not eliminated. Reducing this zone of overlap would automatically reduce the amount of technical biological discussion necessary to explain values falling within this zone. Class I organisms, by definition, are indicative of clean water and are consequently more important in deriving the index value than are Class II organisms. Application of a factor of two to the number of species of Class I organisms appeared both honest and conservative. The formula developed was:

$$2\,(\text{n Class I}) + (\text{n Class II}) = \text{Biotic Index}$$

where (n) is the number of species.

A factor greater than two would not eliminate the zone of overlap and would serve only to increase the range of values that could be occupied by the biotic index. The final calculation was checked against the above-mentioned number of collections and appeared satisfactory.

The "biotic index" may be defined as an index value based on biological findings and indicative of the cleanliness (with regard to organic pollution) of a portion of a stream or lake. It may occupy any value ranging from zero to about 40. The lowest value which may be accepted without discussion as indicative of a clean stream is 10. A grossly polluted stream will have an index of zero, whereas a moderately polluted stream will have a biotic index of 1 to 6. The index of a clean stream with a monotonous habitat and low velocity may range from 4 to 9. Such situations, however, are rare and when they do occur a relatively lengthy explanation is generally necessary.

The following tabulation of the combinations producing a biotic index of 10 will demonstrate why this value has been selected as the lowest accepted without discussion as indicative of clean waters:

Numbers of Species			
Class I	Class II	Biotic Index	Remarks
5	0	10	Clean
4	2	10	Clean
3	4	10	Clean
2	6	10	Clean
1	8	10	Improbable combination
0	10	10	Improbable combination

In the above tabulation, it will be noted that the first four combinations are unquestionably clean. Since there are only ten species of Class II organisms and since many of these organisms differ widely in their habitat requirements, it is highly improbable that eight species would appear together and it would be virtually impossible to find all ten together. As a result, the only probable combinations producing a biotic index of 10 are indicative of a clean stream.

Discussion

Figure 1 demonstrates the usefulness of the biotic index for the presentation of biological results in graphic form. The 1954 curve represents results obtained before a pulp mill began operations. Station 4, located above any possible pollution, serves as a control station. The mill waste is discharged between stations 4 and 19, and the stream flow is from station 4 to 2. Values for the B.O.D. in the polluted portions of the river in 1955 ranged from a maximum of 173 p.p.m. at station 19 to 18 p.p.m. at station 2, a distance of approximately 8 mi.

Certain factors must be considered in the use of this index. Figure 2 shows the comparison of results obtained during a period of low discharge (1952) with a period of high discharge (1953). The behavior of the Escambia River is apparently unique among Florida streams. This river seldom floods in the accepted sense of the word. The difference between "high" and

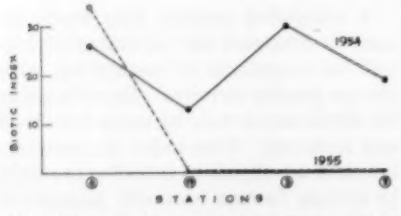

FIGURE 1.—Graphic presentation of biological results using the biotic index.

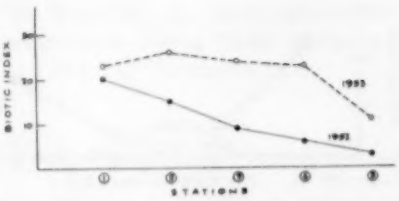

FIGURE 2.—Comparison of the biotic index obtained during high and low flow in the Escambia River, Florida.

"low" water in the Escambia is a matter of increased or decreased velocity.

During periods of low water there is a tongue of salt water which covers the bottom of the river from the mouth to a point above station 3. Since the biotic index is calculated from the presence of freshwater organisms, it logically follows that only those organisms which can tolerate varying degrees of salinity can exist in the lower reaches of this river during periods of low water. As the water velocity increases, the salt tongue is pushed back from the river and freshwater conditions exist even at station 5. There was a significant increase in the biotic index at this station in 1953 due to two main factors. Organisms typical of upstream stations were washed downstream from their more typical habitats, and survived in an area where they could not exist during periods of low flow due to the salinity. At low flow the chloride ion concentration may be as high as 12,000 p.p.m., while during high water it drops to 5 p.p.m. During periods of high water stonefly nymphs and barnacles may be found existing on the same logs at station 5, a sight sufficient to shock the most impassive biologist.

Consequently, increasing salinity produces a definite effect on the biotic index. With all other factors remaining constant, an increase in the velocity of a stream is accompanied by an increase in the biotic index, while lowering the velocity also lowers the biotic index.

In any proposed method, reproduc-

FIGURE 3.—Results of two biological surveys made at the same stations one year apart.

tibility of results is a major consideration. Results of two biological surveys made at the same stations on the Withlacoochee River one year apart are shown in Figure 3. The river is moderately polluted by citrus canning wastes entering the stream a short distance above station 2. The biotic index is lowered considerably at Stations 2 and 3 and shows evidence of the beginning of recovery at station 4. The two curves are close enough to indicate that these methods are reproducible.

The biotic index is calculated from the numbers of species of organisms classified with regard to organic pollution. This basic classification ignores other forms of pollution, of which toxicity is perhaps of greatest importance. Figure 4 shows that a major faunal alteration is occurring on the Alafia River Basin where a rapidly expanding phosphate processing industry is located. The more sensitive species were totally lacking from station GB; a healthy fauna present at station 4 in 1951 had been almost entirely destroyed by 1955; and degradation to a lesser degree is noted at station 20. It is not yet known whether this is due

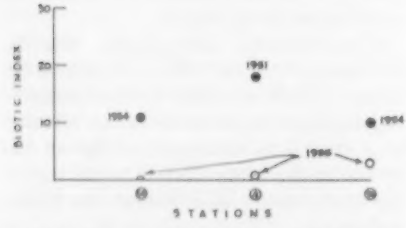

FIGURE 4.—Biotic index indicative of major faunal alteration on the Alafia River, Florida.

to inorganic acids or to the fluoride ion, both of which are present in appreciable quantities. Evaluation of toxicity by means of the biotic index is being studied at present, and it is hoped that bio-assay work will begin shortly.

This index is not proposed as the final word in the methodology or reporting biological results. However, it is a method that has proven especially useful in stream condition investigations in Florida.

The biological methods used have several advantages. They are both rapid and inexpensive. Identification of organisms is a slow task initially, but becomes progressively more rapid as the biologist becomes familiar with the fauna of the area where he works. If these organisms are selected carefully with regard to ecological and geographical distribution, the eventual list of classified organisms need not be a lengthy one. By these methods a clean stream may be proven clean in 15 to 30 min. Inability to prove a stream clean by biological methods constitutes evidence of pollution and polluted conditions require lengthier and more careful sampling since the collector must be certain the sensitive animals are actually missing and have not been overlooked due to haste.

Both the basic classification of organisms and the biotic index are useful throughout the year in Florida where these organisms are present during all seasons. Sampling done in January, therefore, is just as productive as sampling done in June.

Summary and Conclusions

A simplified method (the biotic index) is proposed for reporting biological investigations in connection with stream quality surveys. Reports using the biotic index may be made brief and non-technical. This index is reproducible and is subject to interference only by stream factors normally considered as major environmental alterations by the aquatic biologist.

It is the author's belief that biotic

data are economical; are fully as important as chemical and biochemical data; and have not been more widely used in stream sanitation work because of the extreme wordiness and technicality of biological reports.

Acknowledgment

The author wishes to express his appreciation to David B. Lee and John W. Wakefield for their patience and cooperation during these studies and in reviewing the manuscript.

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PROFESSIONAL OPPORTUNITIES

Ohio—The Ohio Department of Health invites applications from sanitary engineers and sanitary chemists for the following positions: Sanitary Engineer III, water supply development; Sanitary Engineer II, river basin water pollution studies; Engineer-In-Training, river basin water pollution studies; and Sanitary Chemist, river basin water pollution studies. Salary ranges are from $360 to $600 per month based on professional experience and education.

Inquiries should be addressed to Robert A. Ford, Personnel Officer, Personnel Office, Ohio Department of Health, Columbus 15, Ohio.

USPHS—Competitive examinations for appointment of Sanitary Engineer Officers to the Regular Corps of the USPHS will be held on January 10, 11 and 12, 1956. Application forms may be obtained from the Chief, Division of Personnel, Public Health Service, Department of Health, Education and Welfare, Washington 25, D. C. Closing date for applications is Dec. 9, 1955.

THE OPERATOR'S CORNER

CONDUCTED BY DONALD P. SCHIESSWOHL

ACCIDENTAL?

Another accident has occurred in a sewerage installation (reported elsewhere in this Corner). This time the operator suffered second and third degree burns on the exposed portions of his body. Several weeks of hospitalization were required, not to mention the discomfort and loss of work during the post-hospitalization period.

There are undoubtedly many major and minor accidents at treatment facilities that remain unreported due to limited publicity. Every reader of the Journal should take heed from the comments of Robert D. Gidel, Senior Engineer, Industrial Department, National Safety Council:

"Where do they get off calling it an accident when most people get hurt? Looks to me like someone doesn't know what an accident is.

"Reminds me of the time Joe cracked up his new convertible. Papers said, 'the car went out of control on a curve.'

"Joe didn't lose control of the car—he never had it! I happen to know Joe was half asleep at the time, yet they called it an accident. The only accident was that he wasn't killed. Me too, because I was with him.

"Then there was the day my next-door neighbor fell off his 20-foot extension ladder. He had one foot of it on the ground and the other foot on the concrete sidewalk. He reached out to the side a little too far to paint a shutter, and blooie! Right in the middle of his evergreens.

"They called it an accident too, but it looks to me like the only accidental part of it was that he didn't break his neck as well as his arm.

"One day on the job Milt had to do a little repair work on a compressor. As usual, he had his rear pockets jammed with tools, including a 12-inch screwdriver. As he was crossing the storage yard he tripped over a piece of 2-inch pipe and fell, ramming the tip of the screwdriver about an inch into his back.

"Yep, they called it an accident. But, I still say it was merely accidental he didn't get a permanent case of sclerosis of the liver, 'er something.

"Looks to me like somebody is way out in left field when they call such things accidents. I thought an accident was a chance happening, unexpected, unintentional and without known or assignable cause. Didn't all these things have causes? Couldn't they have been expected? But, who am I to speak up against all these experts on the subject.

"Seems to me if they would quit calling such things accidents, maybe guys like me would think more about not having them.

"Just like old man Murphy, who blew himself up using some flammable solvents in the basement near his gas heater.

"Man, what an accident it would have been if he hadn't been killed!"

You and only *you* are responsible for your own safety in situations like the ones mentioned above. So, for your own sake and those dependent upon you—remember Mr. Gidel's comments before you undertake work around the treatment plant or a do-it-yourself job at home.

D. P. S.

PROBLEMS OF OVERLOADED SEWAGE TREATMENT PLANTS *

By Harry A. Tow

Director of Public Works, Santa Barbara, Calif.

Once every 20 years more or less, every operator and every city or sanitary district is faced with the problem of an overloaded sewage treatment plant. Exactly when this problem occurred last or when it will occur again depends on a number of factors—community growth, engineering foresight or effective plant operation. It may be occurring now. Each time it recurs, the community reluctantly faces the probability of financing new treatment facilities in order to prevent pollution.

There are many treatment plant operators who can operate a properly designed sewage treatment plant functioning at less than design capacity, although a high degree of skill is required to operate the plant most economically. The true value of the capable treatment plant operator or administrator is determined by his ability to deal with a major problem such as presented by a basic overload of the treatment facilities. He must be able to:

1. Recognize an overload problem, preferably while it is still developing.
2. Initiate effective studies directed toward correction, by determining the basic treatment units involved in the overload.
3. Recommend or take a course of action, while concurrently making a basic decision as to whether independent action is sufficient or whether a consulting engineer's opinion should be obtained.
4. Convince the financing authorities, if expenditures of funds are involved, of the necessity for action, and effectively present the consequences of delay.

Basically, the operator or administrator has two paramount responsibilities in the analysis and solution of an overload problem: (a) the prevention of hazards to the public health caused by pollution; and (b) the provision of treatment adequate for this purpose at the lowest possible long-term cost to the community he serves. These responsibilities, though shared to some extent by consulting engineers, community officials or responsible state control agencies, can be most objectively analyzed by the sewage treatment plant operator or administrator. If well trained and completely impartial in his analysis of an overload problem, he is better able than any other individual to solve the problem effectively and economically.

Indications of Overload

Treatment plants do not automatically become overloaded when the flow reaches and passes the magic number the design engineer has designated as "design flow." Actual overload of individual process units may occur well before design flow is reached, or, design flow may be greatly exceeded before any real overload exists.

Regardless of design criteria, a plant may be said to be overloaded when it is no longer economically producing effluent and sludge sufficiently stable to permit easy disposal without a health menace. It is not always necessary for the trained operator to wait until he has a continuing high coliform count in the receiving waters or drying beds full of sour sludge to recognize an incipient

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

overload. The symptoms of approaching difficulty are cumulative and easily recognized if plant performance is checked regularly. Increased suspended solids in the effluent, increased digester volatile acids, decreased freeboards during storms and sharply increased daily flows present undeniable demands for early corrective action.

When such evidence accumulates, the operator should not await surveillance by the state water pollution control authority or complaints from downstream water users before recommending consideration and study of the overload problem. The historic inertia of some elected officials in dealing with such problems and the operator's personal welfare in being first to "sound the alarm" both dictate the wisdom of advising governing bodies of the problems they are facing or are soon to face. This is true even if the proper immediate solution to the overload problem is increased operating costs rather than capital expediture for new plant and equipment. It is wise for the operator to explain the probable consequences of failure to act conclusively in terms of punitive action by regional or state authorities, and/or the restriction of the community's development because facilities are inadequate.

Analyzing the Problem

In considering a course of action with regard to an impending or existing plant overload, the operator should attempt to isolate the process units actually overloaded from those continuing to serve satisfactorily, or which would serve satisfactorily if other units were not malfunctioning. In some cases, the units involved are perfectly obvious; in other cases, careful sampling and checking, consideration of unit design capacities and existing process flows, and plant scale experimentation may be required to isolate the offending units or to determine that all are at or approaching capacity.

Depending upon the experience of the operator, and the amount of time and qualified technical or engineering assistance available, the supervisor may require the services of a competent consulting engineer to aid in analyzing the overload problem. Even if this is the case, it is well for the operator to study the problem carefully, since a consultant cannot hope to become as familiar with plant operations as the operator himself. Consequently, he must rely on the operator for a large part of the basic information. The selection of a consultant should be made carefully on the basis of known abilities in the sewage treatment field and experience with operating problems.

Plant overload problems fall into one or more basic categories under which solutions may be logically grouped:

1. Sewage treatment problems.
2. Sludge treatment and disposal problems.
3. Sewage flow hydraulic problems (freeboard and pump capacities).
4. Industrial waste problems.

Having determined the nature and extent of an overload problem, the plant operator or administrator must determine alternate methods of correction and select one. If the selection made does not include major plant expansions or capital investment, the services of a consultant may not be required. On the other hand, if a considerable capital investment is required or if uncertainty exists as to the best method of attacking the problem, a reliable consultant should be engaged. The consultant can assist in economic analyses of proposed process changes or equipment expenditures, can aid in "selling" the need for action to legislative officials and can take the brunt of any criticisms of a major and costly enlargement program.

Solutions for Overloads

Measures short of major plant expansion that are commonly used and which should not be overlooked by the

operator, either as temporary expedients or as long term solutions, include process changes, operation modification and control, control of plant influent characteristics and minor equipment replacements. Examples of possible solutions to basic overload conditions may serve as the first step in the adoption of an objective engineering approach to a solution of an overload problem.

Sewage Treatment Problems

1. Chlorination.—A temporary expedient at best, increased post-chlorination may permit continued satisfactory receiving water conditions in terms of coliform count when dilution alone will no longer suffice for treatment of an unstable effluent from an over-loaded facility. The continued cost of increased chlorination must be carefully balanced against the capitalized cost of more permanent improvements and the point of diminishing return noted. Sludge deposits, excessive algal growths and other problems sometimes accompanying discharge of an unstable effluent will not be greatly improved by increased chlorination.

2. Oxidation ponds.—If land is available, oxidation ponds may be utilized for polishing an effluent that is unstable after treatment in existing process units. This is particularly true of inland communities with high yearly evaporation rates. Initial investments are low if land is available. Units of this type should be a first consideration for plant expansion where proper climatic conditions and available land permit.

3. Outfall modifications.—Staffs of treatment facilities incorporating ocean outfalls as an integral part of the process may often greatly increase the efficiency of plant operations and forestall process unit expansion by careful study of lengthening the existing outfall facilities or installing modern diffuser sections for increased dilution. Common practice in outfall installations in the past dictated installation of the outfall pipe at depths that could be most economically reached. Less consideration was given to adequate dilution or depths of submergence as is now possible as a result of increased knowledge of outfall capabilities.

4. Chemical precipitation.—As a temporary expedient where complete treatment plant expansion cannot be made when required, chemical precipitation will sometimes afford a reduction in suspended solids and B.O.D. Continued use of chemical precipitation, however, generally will prove completely unsatisfactory from a cost standpoint.

5. Elimination of recirculating solids loads.—A common source of difficulty when treatment capacities approach or exceed maximum design capacities is the imposition of a heavy recirculating solids load upon the clarifier facilities. It is often better at this stage in the treatment plant life cycle to avoid solids recirculation and discharge directly to the outfall facilities with adequate chlorination to minimize effluent instability. Continued recirculation of the solids load serves only to affect biological treatment processes adversely and to increase the B.O.D. of the effluent prior to final discharge.

It is essential that proposed changes of this nature be carefully reviewed with state or area health and pollution control authorities.

6. Process changes.—The most feasible method of effecting postponement of major plant expansion is a change in treatment processes involving reductions in the degree of treatment for which the plant was originally designed. These process changes may result in a higher degree of treatment than what the plant can produce using the original design processes at excessively high rates of flow. Examples are:

a. It has been found possible in some cases to increase plant treatment capacity by large percentages by elim-

inating preaeration facilities that have become inadequate and utilizing the tank capacity thus released for primary settling. At the Santa Barbara, Calif. treatment plant total plant capacity was increased from 6.4 m.g.d. to approximately 8 m.g.d. by such change. This change, although causing a negligible increase in suspended solids, increased the B.O.D. reduction and decreased the final effluent sulfides.

b. In fairly large plants, where adequate laboratory control can be maintained and where thorough experimental programs can be carried out, considerable success may be achieved in enlarging plant capacities economically by substituting one of the other aeration processes for standard rate aeration. Although a decrease in treatment effectiveness up to 10 per cent may be caused by such a change, many receiving waters are adequate to complete the treatment process of these relatively well stabilized effluents with only a minimum increase in the chlorination rate. It should be pointed out that several of these aeration processes are extremely delicate and require controls which may demand more time and facilities than are available at small plants.

c. Where filter media and clarifier capacities are adequate, a considerable increase in standard-rate trickling filter plant capacities can be obtained by installing the necessary recirculation pumps and piping, thus converting standard-rate filters to high-rate trickling filters. Once again, a slight decrease in treatment efficiency over design efficiency for standard-rate filters may be anticipated. However, actual operating results may be better than the year-round results obtainable with standard-rate filters, since sloughing and similar ills are largely nonexistent with high-rate filters.

Sludge Treatment and Disposal Problems

Modern digestion processes presently being developed have made it possible to increase greatly the efficiency of digestion in existing tanks, thus providing additional capacity without major plant expansion.

The new patented processes, as well as the nonpatented operating methods now in effect in some of the large plants, all emphasize increased and more efficient recirculation of solids by means of pumping, gas flotation or similar methods. In addition, careful feeding of raw sludge at controlled rates and solids percentages has demonstrated its effectiveness in increasing digestion efficiency. The net result of the experimental work has been a basic revision in current theories of digestion efficiency with proper control.

Increased digester capacity can, in favorable locations, be obtained by the use of open tanks for completion of the secondary digestion process. Gas production is virtually completed in properly operated primary digesters so less expensive open secondaries will cause few odor or hazard problems, except in congested areas. Any installation of this type must be carefully reviewed with the controlling health and pollution authorities before initiating construction. The design of open digesters should also consider whether or not secondary digestion or sludge concentration is the desired result.

Careful control of the digester scum blanket is essential prior to consideration of the adequacy of digester capacity. Reductions of up to 50 per cent in digester efficiency can be caused by failure to remove scum blankets that have built up over a period of years. Digester scum should not be permitted to occupy excessive percentages of tank volume.

The immediate reaction to digester difficulties of any kind is that a lack of digester capacity is the probable cause. This assumption should be carefully checked by laboratory analysis of digester conditions over an adequate period of time, to assure that sludge digestion cannot be measurably improved

by an increase in the efficiency of normal operation and control measures. The digester capacities installed in most treatment plants may be adequate for some time if the necessary piping, pump modifications and operational controls can be established in accord with modern practice.

In some cases sludge disposal facilities designed for small treatment plants have proved inadequate as a result of changes in the fertilizer market or the existence of difficulties in sludge treatment not envisioned in original design. For example, sludge filtration plant facilities sometimes installed in small or medium sized treatment plants may prove difficult to operate at a reasonable cost. Actual abandonment of these facilities in favor of sludge drying beds, hauling of liquid sludge to nearby farm lands or sludge disposal through ocean outfalls may sometimes be necessary. In other cases minor modifications of installed facilities may render them usable.

Changes in local market conditions for fertilizer may require increases in sludge drying facilities or conversion to the disposal of liquid sludge. The treatment and disposal of digested sludge is not subject to analysis by general precept, but must be considered individually for each plant with relation to the facilities and market areas available.

Sewage Flow Hydraulic Problems

Treatment plants are sometimes faced with a lack of hydraulic capacity. Inadequate free board during storm flows or daily peaks may be the controlling factor in the timing of necessary expansions. Several methods are available for control of total sewage flows; all should be carefully evaluated before additional units are built for this purpose alone. Among these methods are:

1. Reduction of sewer infiltration during storms, by prohibiting storm drain connections and eliminating joint infiltration and manhole cover leakage. A policy of public education accompanied by adequate publicity and spot inspection may result in reducing illegal storm drain connections to the sanitary sewer. Success generally can be obtained in reducing flow through manhole covers by closure of excessive openings and by replacement of known sections of badly deteriorated line in areas with high ground water tables.

Excessive dry weather flow is sometimes caused by the connection of cooling system waste lines to the sanitary sewer system. A program of education and the passage of a formal ordinance requiring recirculation of cooling water and prohibiting discharge directly to the sewer system may have a significant effect upon the total amount of flow to be handled.

2. Reduction of pump surges by installing variable speed pumps so excessive pump surges are eliminated and plant flow systems can thus handle increased storm flows without increases in basic flow capacities.

3. Separation of industrial wastes where heavy hydraulic demands are made on a sanitary sewer system and treatment plant. It may be possible to treat the wastes separately or discharge them without treatment and thus achieve more effective plant capacity.

Industrial Waste Problems

In addition to the possibilities of separate disposal of industrial wastes, many operators may find on careful analysis that industrial wastes are a major factor contributing to overload. Two basic solutions to industrial waste overload problems are: (a) adequate pretreatment requirements and (b) the enforcement of controls on the types of wastes that may be discharged without pretreatment. Sludge digestion, activated sludge processes in small plants, and other biological processes can be badly disrupted by continuous

or intermittent discharge of industrial wastes on an uncontrolled basis. Even though many communities have industrial waste ordinances, enforcement policies in most small communities are notoriously lax. Any consideration of treatment plant expansion should be preceded by a careful analysis of the industrial wastes and whether or not adequate treatment may postpone plant expansion.

Summary

An overloaded sewage treatment plant may not necessarily require major plant or process unit expansion. Careful analysis and effective action by plant personnel can often solve overload problems by segregating these problems into:

1. Sewage treatment problems.
2. Sludge treatment and disposal problems.
3. Sewage flow hydraulic problems.
4. Industrial waste problems.

By applying process conversion, flow control, equipment modification, and process control methods the problems thus segregated may be resolved. Improved treatment at minimum cost may be obtained more economically, and maximum service to the community assured.

HOUSE CONNECTIONS—WHOSE RESPONSIBILITY?*

By Cedric Webster

Superintendent, Menlo Park Sanitary District, Menlo Park, Calif.

Almost 4,000 years ago King Hammurabi of Babylonia invented a sewer pipe that surprisingly enough looks like its 20th Century counterpart, having the same general size and shape. It even incorporated a modified version of the bell and spigot joint and was made of clay.

While a study of history will confirm the accuracy of the foregoing, it fails to disclose what Hammurabi said when his brand new sewer backed up and flooded the royal bathroom. There can be no doubt that the line did back up for in spite of the best efforts and best brains that have been expended in solving the problem of the obstructed sewer, the problem is just as real today as it was then.

The pipe, jointing material and laying technique have been improved, but the problem of how to prevent the sewer from becoming obstructed is still unsolved. Of all the parts of a sewer system, from the kitchen sink to the secondary clarifier, there seems to be nothing so apt to give trouble as the house connection. Whatever may be the immediate cause of the obstruction; be it roots, faulty joints or an old undershirt, it is this connection where trouble seems most likely to occur. The prevalence of this trouble seems to arise principally through improper or faulty installation, rather than anything inherently defective in the materials used.

The materials used in house connections are also used in the construction of the collector sewers themselves. The chief difference apparently is that sewers are generally designed and constructed by competent professional engineers and contractors who work under favorable conditions, and the lines presumably receive careful inspection. The house connection is all too often the indifferent product of a slipshod plumber. A plumber who might take

* Presented at 1955 Annual Meeting, California Sewage and Industrial Wastes Assn.; Riverside, Calif.; April 27–30, 1955.

real pride in the quality of his work insofar as the house plumbing is concerned, may all too often look with anything but satisfaction on the house connection he has made. It is no wonder that this is true because cutting a trench in a busy street and making the sewer tap is at best a miserable job. What with the street department clamoring for him to close up the trench and the fire department screaming that they can't get their trucks through, it is small wonder the poor plumber has but one thought—cover up the connection and let the property owner or the sewer department worry later.

In some cities and sanitary districts the responsibility for making the house connection is that of the sewer department. This, it would seem, is the proper way to operate and the best way of assuring that a good connection is made every time. Fortunately, at least in certain areas of the state, the sewer tap is fast becoming a thing of the past. There are not many vacant lots left in the older subdivisions and in the newer ones the house laterals are run to the property line at the time the collection system is constructed. Quality construction, under the conditions prevailing at the time, is the rule rather than the exception. As a result, the amount of maintenance in the newer subdivisions is considerably less than in the older ones.

Whose Responsibility?

When a main line sewer becomes obstructed, it is without question the responsibility of the appropriate public agency to correct the trouble. When the kitchen sink refuses to drain the property owner recognizes that it is either his or his plumber's problem. But when the obstruction is in the house connection, and more properly when it is between the main sewer and the property line, whose responsibility is it then?

Some cities and sanitary districts take the attitude the house lateral is the exclusive property of one or more individuals, therefore public funds cannot be expended in maintaining it. Other areas will provide maintenance only upon payment of a fee or charge for the work. In still other sections, the responsibility is assumed by the public agency and all maintenance is provided without charge. No one can question that the house lateral belongs to the owner of the property it serves; it is his by right of installation or purchase. This being the case then how can the expenditure of public funds for its maintenance be justified? Why should a public agency be called upon to remove some foreign object such as a diaper which through carelessness has been admitted to the house plumbing and becomes lodged in the house connection?

If all sewer obstructions were the result of carelessness, the question of responsibility would be simple. Suppose, however, that the obstruction is caused by some event over which the property owner has no control. Possibly the water company or the gas company has damaged the line. Perhaps roots from the main line sewer have entered the lateral or it has been crushed or damaged by heavy vehicular traffic. These are all common causes of trouble over which the property owner has no control. Then too, consider the difficulties confronting an individual if he is expected to cut a public street in order to repair the damaged lateral. The problem is not only difficult but expensive.

Policies

For 47 years the Menlo Park Sanitary District followed the policy of refusing all maintenance of sewer laterals. Experience under this policy had not been satisfactory. Explaining to an irate taxpayer just why the house connection serving his property cannot be serviced can be a trying experience.

The policy of making a charge for the maintenance of house laterals would

put a premium on the ability of the individual to pay and would not be in keeping with the public interest. Furthermore, where the property owner honestly could not afford to pay for the work the public agency would act in order to avert a menace to the public health. In some areas the policy is to charge for lateral maintenance when it can be determined that the obstruction is caused by some negligent action on the part of the property owner. Such a policy might well require the wisdom of Solomon in order to make a fair determination and seems unrealistic at best.

For the past 7 years the Menlo Park Sanitary District has followed the policy of providing maintenance for the house connection between the property line and the sewer. This service is provided to all property owners without charge and has been entirely satisfactory. The property owner must provide a clean-out at or near the property line. With a clean-out it is generally the work of only a few minutes time to clear an obstruction. When it is necessary to perform a "dig-up" job this service is also provided without charge. Those portions of the house lateral lying within a public street or easement are considered, maintenance-wise at least, as a part of the public sewer system and are treated accordingly.

The cost of providing this additional service has not been a burden on the taxpayers as reflected in the tax rate which has been cut from $0.36 per hundred dollars assessed valuation in 1951 to the present rate of $0.19. To date complaints have not been received about using public funds for the maintenance of private house connections, nor are they anticipated. The public seems genuinely appreciative of what is being attempted. Public maintenance of house connections is something which has worked well for Menlo Park and is heartily recommended.

CLASSIFICATION PLAN FOR CALIFORNIA OPERATORS

The purpose of classification for positions in government or private industry is to define the duties and responsibilities of each position so adequate information is available to management for the development of a proper organizational structure. Classification is the first step toward logical organization and is the basis for job and wage improvement. Confusion and vagueness are eliminated by specifications developed through job investigation, employee surveys and work questionnaires. Positions involving similar duties and responsibilities may be consolidated under one job title thereby providing simplification of titles, yet including adequate flexibility. Classification may be described as an organizational tool useful in establishing rank of authority, wages and promotions, and as a recruitment aid.

The scope of this classification plan has been limited to the preparation of tentative job descriptions for various grades of operators. The problem was to reconcile the conflict of titles, duties and responsibilities into a few simple job descriptions for an operator that would be all inclusive yet workable and satisfactory to the operators. The primary objective is to arouse interest in the use of classification as a means to improve the position and salary of the operators.

The outline of a typical job description consists of five parts:

1. *The Title*—Great importance is attached to the title of a position—unduly so in many cases. Employees may take pride in the title or, if it is not to their liking, it may engender dissatisfaction and unsatisfactory work at-

titudes. A variety of titles were found in current use, such as: plant operator, sewage plant operator, treatment plant operator, sewage treatment plant operator and sewage treatment works operator. In consideration of the past practice of the operators group and more particularly the recently proposed Sewage Treatment Works Operator's Act (Calif.) the title "sewage treatment works operator" was selected as most acceptable to the operators group. The title is certainly descriptive enough; whether it is a "works" or "plant" is of little consequence.

2. *Job Definition*—A general statement in short form defining the principal job functions performed, including the extent of the supervision received or exercised. Since neither the definition nor the entire job description are restrictive the clause "does related work as required" is included to cover special assignments or emergency situations.

3. *Distinguishing Features*—This section defines the level of difficulty and responsibility of each classification so the characteristics may be readily distinguished from each other. This is particularly important when classes bear the same title.

4. *Examples of Duties*—A list of typical functions or operations performed by an incumbent, reflecting the

TABLE I.—California Operators Classification Plan

Item	Title		
	Operator I	Operator II	Operator III
Definition	Under general direction of superintendent. Plans, organizes and supervises work.	Under general supervision. Works with and directs group operations.	Under direct supervision. Operates equipment, makes minor repairs.
Features	In complete charge of operations.	Responsible for proper operation of equipment and execution of standing orders.	Operates equipment under direct supervision.
Duties	Prepares annual budget, makes inspections and recommendations, issues instructions, personally assists and directs difficult operations.	Supervises and instructs operators, keeps log and time sheet, makes chemical tests and submits routine reports.	Operates equipment and makes minor adjustments, reads meters, keeps records and maintains buildings and grounds.
Knowledge	Thorough knowledge of principles and current practices of sewage treatment.	Good knowledge of treatment processes and scientific principles involved.	—
Ability	Ability to plan, organize, direct, review and coordinate work.	Ability to effectively supervise and instruct subordinates.	Ability to operate equipment, read meters, use tools.
Experience	4 yr. experience in responsible charge.	2 yr. experience, full time.	One year semi-skilled experience with tools.
Education	High school graduate plus 2 yr. college. Qualifying experience may be substituted for college.	High school graduate. Do arithmetic and simple chemistry.	2 yr. of high school, ability to read, write and do simple arithmetic.

nature, difficulty and responsibility of the duties of the position. All duties cannot be included nor is it essential that all duties so included actually be performed.

5. *Minimum Qualifications* — Minimum qualifications comprise a most controversial section and one which will probably cause more dissention than any other part. Knowledge and ability of the job must be demonstrated by written and/or oral examination. Therefore, the statement contained in the job description may be considered as the basis for examination. Further demonstration of these qualifications is provided in the customary probationary period following appointment.

Experience requirements are factors that depend upon the difficulty of the work, the responsibility, the training period required to become proficient in all the elements of the next higher position. In addition, sufficient time is necessary to become acquainted with the requirements of the position through training, education or actual performance of the duties under substitute or temporary appointment. Even the lowest classification of operator is considered a skilled position and common labor should be classified under a title of lower responsibility.

Educational requirements are a frightening hurdle for many aspirants to advancement and for this reason are kept to a minimum. A low classification will preclude salary increases while an upgrading of the position will open the way for advancement.

The Committee was aware of many imperfections in the classification plan, which is in a transitionary stage, and that state licenses for operators is the current objective. Classification as herewith presented should not conflict with this objective. Perhaps the title "Superintendent" should be reserved for those in charge of large installations in big cities where engineer training may be required. In many small cities the city engineer or street superintendent is actually the sewage treatment works superintendent since he exercises administrative jurisdiction.

Tabulation of the qualifications for the three grades of sewage treatment works operators is given in Table I.

INTERESTING EXTRACTS FROM OPERATION REPORTS

"Few things compare with the satisfaction of a job well done."—*Anon.*

Attention operators! It will be appreciated if copies of all annual reports of both large and small plants are sent to the Federation office for abstracting in this section of the Journal. Carbon copies of typewritten reports will be promptly returned on request. Please direct annual reports to: Federation of Sewage and Industrial Wastes Associations, 4435 Wisconsin Ave., N.W., Washington 16, D. C.

Annual Report of the Galesburg (Ill.) Sanitary District Sewage Treatment Plant (1953 and 1954) *

By L. W. Hunt, *Superintendent*

Approximately 100 m.g. less sewage was treated in 1953 as a result of below average rainfall. The average strength was 24 per cent greater. Flow in 1954 showed a decrease of 6 per cent over the preceding year and was accompanied by an increase in sewage strength of 10 per cent. There was no

* For last previous report see This Journal, **25**, 7, 855 (July, 1953).

TABLE I.—Summary of Operation Data for 1953 and 1954 at the Galesburg, Ill., Sewage Treatment Plant

Item	Average 1953	1954
Connected population	33,000	34,000
Equivalent population (B.O.D.)	43,000	36,379
Rainfall (in.)	24.62	37.15
Interceptor flow (m.g.d.)	5.66	5.41
Sewage receiving primary treatment (m.g.d.)	5.09	4.81
Sewage receiving secondary treatment (m.g.d.)	5.09	4.81
Screenings (cu. ft. per m.g.)	0.47	0.4
Grit removed (cu. ft. per m.g.)	2.35	2.4
Scum removed (cu. ft. per m.g.)	1.79	1.3
pH raw sewage	7.4	7.5
pH final effluent	7.4	7.5
Suspended solids:		
Raw (p.p.m.)	170	179
Settled (p.p.m.)	87	88
Filtered (p.p.m.)	83	78
Final (p.p.m.)	33	28
Reduction (%)	79.5	83.8
Raw (lb./cap./day)	0.21	0.21
B.O.D.:		
Raw (p.p.m.)	176	194
Settled (p.p.m.)	132	140
Filtered (p.p.m.)	71	64
Final (p.p.m.)	38	34
Reduction (%)	77.8	82.2
Raw (lb./cap./day)	0.22	0.26
Filter loading:		
B.O.D./acre ft. (lb.)	443	474
B.O.D./1,000 cu. ft. media (lb.)	5.5	10.8
B.O.D./cu. yd. media (lb.)	0.15	0.29
Primary sludge:		
Total solids (%)	7.1	7.5
Volatile solids (%)	62.9	61.7
Secondary sludge:		
Total solids (%)	1.5	1.4
Volatile solids (%)	54.1	53.4
Digested sludge:		
Total solids (%)	8.8	8.8
Volatile solids (%)	44.4	43.5
Gas production:		
Cu. ft. per lb. Total solids to digester	4.95	5.03
Cu. ft. per lb. Volatile solids to digester	7.82	8.16
Cu. ft. per m.g. raw sewage	5,447	5,649
Operating costs ($ per m.g.):		
Gas engines	0.29	0.57
Payroll	17.27	19.29
Power	0.69	0.70
Trucks	0.26	0.36
Laboratory supplies	0.07	0.06
New tools and equipment	0.36	0.44
Repairs	2.15	1.50
Office expense	0.11	0.12
Insurance	0.56	0.62
Care of grounds	0.19	0.24
Miscellaneous	0.34	0.48
Total	22.29	24.38

immediate accounting for the continued decrease in flow.

Treatment efficiency was approximately 5 per cent better in 1954. All sewage received was treated by the facilities of this 6 m.g.d. trickling filter plant.

Dry solids removed totaled nearly 1,000 tons, and gas production was approximately 9,600,000 cu. ft. Operation of the gas-engine pumps continued to result in a net saving in power cost of over $3,000. Digested sludge was handled in the following manner:

55 per cent hauled wet to pasture lagoon.
12 per cent discharged to lagoons and dried.
33 per cent discharged to drying beds.

Routine maintenance of equipment consisted principally of cleaning, painting or overhauling. A new impeller was installed in 1953 on the sewage lift pump.

Plant operating data for 1953 and 1954 are summarized in Table I.

PLASTIC JOINT FOR CLAY SEWER PIPE

Improved methods of joining clay sewer pipe have been under study for several years. One of the developments to facilitate the job, which is often time consuming and laborious, is a premolded plastic ring.

Completely root-proof, watertight, clay sewer pipe lines are now possible with new vinyl chloride polymer* molding compounds that permanently seal joints between lengths of pipe. The new compounds are supplied in liquid plastic form to clay sewer pipe manufacturers who process the mate-

*"Koroseal" brand polyvinyl chloride, manufactured by B. F. Goodrich, Inc.

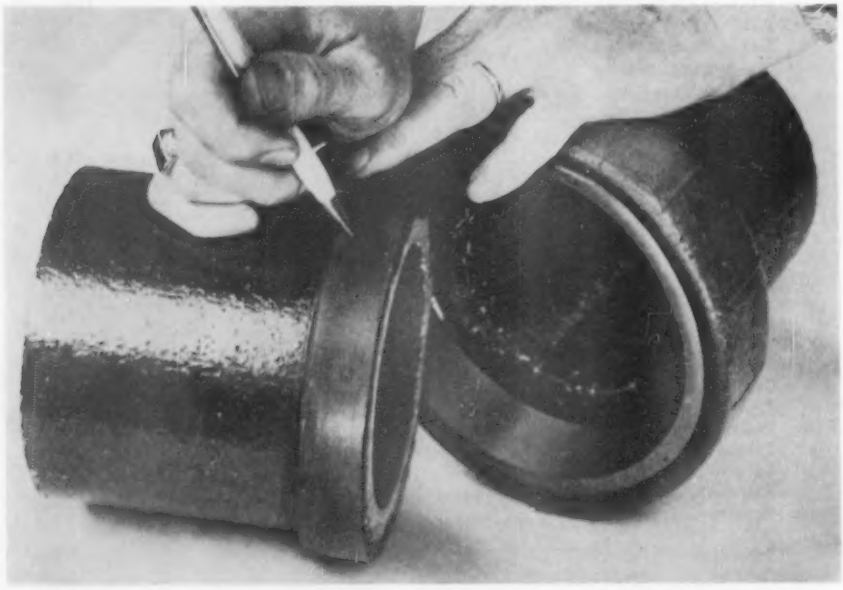

FIGURE 1.—Plastic rings molded permanently to the bell and shaft ends of clay sewer pipe.

rial into tough but flexible sealing rings.

The finished joint is not affected by chemicals, alkalies and acids encountered in underground service. For ease of installation, the molding materials are made to remain flexible at low temperatures, yet firm in hot weather.

Development of this polyvinyl chloride compound was undertaken by B. F. Goodrich in cooperation with leading manufacturers in the sewer tile industry who created new manufacturing techniques to produce the new seals economically.

Contractors receive the new pipe with rings of the plastic material molded permanently at the ends of each pipe section (Figure 1). At the shaft end of the pipe, a ring is molded to the outside surface; at the bell end of the pipe, a ring is molded to the inside surface of the bell. When the two ends are pushed together, the rings squeeze tightly against each other to form a mechanical, compression seal that is reported to remain effective far longer than any other known pipe-joint sealant.

This new pipe jointing method should reduce costs by simplifying installation. Only a few seconds are required to join pipe with a permanently effective seal. Material and labor costs involved in caulking joints by conventional methods are reduced. In addition, good joints are assured under difficult construction conditions.

Clay sewer pipe manufacturers licensed to use the new rubber joint are producing pipe so equipped in diameters of 4-, 6-, 8-, and 12-in., with plans to make pipe up to 36-in. in diameter.

PERFUMED WASTES *

By O. Davenport and B. Woods

Avon Products, Inc., Suffern, N. Y.

Perfumed odors from a sewage treatment plant and still there were complaints. Sewage treatment facilities have been blamed, rightly or wrongly, for noxious odors but not often for sweet odors. This complaint originated at the primary sewage treatment plant of the Village of Suffern, N. Y.

Occasional trouble had been experienced due to emulsified fat which caused trouble in separating the solids from the water. There never was a complaint of perfume odor, as this did not affect plant operation.

The complaint started when the North Jersey Water Commission was granted permission to pump water from the Ramapo River at Pompton Lake to their Wanague Reservoir. A sweet odor was noted in the water. Odor tests were run and the sweet odor was traced from Pompton Lakes up the Ramapo River to the Suffern treatment plant where industrial wastes from Avon Products, Inc. were tagged as the source of the odor.

At this time, Avon Products, Inc. was discharging the wash water from manufacturing kettles and filling machines into the Suffern sanitary sewer. This wash water consisted of cosmetic creams, lotions, bath oils, toilet waters and cosmetic ingredients, many of which form stable emulsions even after considerable dilution.

To establish the quantity of wash water requiring deodorization, the wash water lines were separated from the

* Presented at 5th Joint Meeting, New York Sewage and Industrial Wastes Assn. and New England Sewage and Industrial Wastes Assn.; Albany, N. Y.; June 13–14, 1955.

sanitary sewer. The wash water was collected in two tanks where it was measured and sampled to determine the average composition and quantity of the waste.

After many laboratory experiments, including the running of the threshold odor value, a treatment procedure was adopted. The waste water is gathered in one tank, and the pH adjusted to 10.5 by the use of lime. Aluminum sulphate is added until the pH is lowered to 5.5 and a floc is formed which is removed in an air flotation unit at a flow rate of 1,000 g.p.h. Activated carbon is added to the clarified waste and agitated overnight. The waste is then filtered through a filter press, using diatomaceous earth as a filtering aid, while a holding tank receives the filter effluent. A check is made for odors and if satisfactory the effluent is discharged to the Ramapo River. There have been no further reports of odor complaints.

The foregoing description makes the work pretty cut and dry. However, the segregation of the industrial wastes was a major project. The determination of the amount of waste water per day required considerable study so that a maximum flow could be established and safety factors determined. After the facilities were started the water collected on Friday had an odor that permeated the entire plant by the following Monday. The odor was destroyed by the use of hypochlorite. An average flow of 4,000 g.p.d. is treated. Daily chemical cost is $11.32 and the labor cost is $15.64. Cost of the equipment and building was approximately $45,000.

TIPS AND QUIPS

Another Gas Explosion

Once again a leaking gas main created explosive conditions in a sanitary sewer system—this time in a lift station at Imperial, Neb. As reported by Mearl Orrell, the superintendent, gas leaks had occurred at various locations in the city gas main over a period of three months. Little attention had been given to the trouble until a large leak resulted in gas accumulating in the lift station wet well.

Unaware of this gas, Orrell entered the lift station to service the equipment. Noticing no odor of gas he descended into the wet well. When a moisture proof plug was detached a spark resulted and the interior of the wet well was enveloped in flame. Scrambling up through the fire the superintendent was blown out of the wet well by another and larger explosion. Quickly he rolled on the ground, putting out his flaming clothing. Second and third degree burns on exposed portions of his body required several weeks hospitalization.

When will responsible officials learn to provide protection for operating personnel by furnishing adequate equipment, sufficient assistance and requiring periodic inspection and prompt repair of sewer systems? This question may not apply to Imperial, but it is a serious one in too many sewer systems.

Detergent Grease

Problems of detergents in sewage treatment plants have been discussed pro and con, but not much has been reported on the effects of detergents on sewer systems and pumping facilities. A recent inquiry received at Federation headquarters reported "detergent grease" problems in sanitary sewers and lift stations. Grease plugs 8-in. to 10-in. long in the 6-in. and 8-in. laterals have presented maintenance problems. Removal by rodding or flushing is quite difficult.

An appeal is made to operating personnel and others who have any ideas or experiences on this problem. Please send your comments to the Operator's Corner.

Reflective Signs

Signs offer visibility and immediate identification during darkness when flashlight beams strike the reflective surface made of "Scotchlite" brand reflective sheeting. The colored glass bead sheeting is backed by non-corrosive aluminum, plywood or other materials.

Use of the colored signs aids in quick spotting of first aid equipment, fire extinguishers, tool lockers, exits, telephones or explosive equipment. There is no limit to ingenious uses for the sheeting. Overhead structures, narrow doorways or steep stairways can be outlined to call attention for caution—in darkness or light.

Flexible Nipple

Where piping misalignment, moderate vibration or pipe movement due to expansion and contraction is a problem an all bronze-brass flexible nipple is available. The flexible portion of bronze seamless tubing has deep annular convolutions that permit easy stretching, bending or compressing to fit available space. Standard units in a wide range of sizes have a union fitting on one end and a male fitting on the opposite end. The nipples are available through plumbing supply wholesalers.

One-Piece Hose Clamp

A circular clamp with U-shaped folds or lugs can be slipped over a piece of hose and quickly crimped in place. The parallel sides of the lugs are squeezed together with a pinchers. Lasting and positive clamping action is reportedly assured under extreme internal hose pressure, hose movement and tough usage. Simply snipping off the lugs removes the clamp.

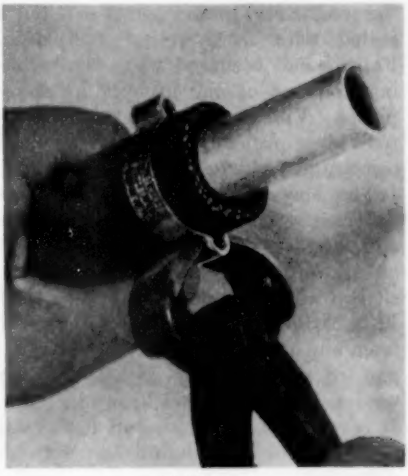

FIGURE 1.

Every clamp is marked with the minimum and maximum diameters of the hose or tube for which it is applicable. Economical for field use, the clamp is available in sizes from ¼-in. to 2 ⅝-in. Supplier is Circle Clamp Corp., 39 Broadway, New York 6, N. Y.

Soak It Up

Accidental grease, paint, or water spillages around the sewage treatment plant can be quickly cleaned up using an all-purpose mineral floor absorbent. According to a recent release The Tru-Sorb Corp., Bridgeport, Conn. claims improved absorptions of 25 to 200 per cent. The new absorbent is non-combustible, harmless to clothing and floors, and is chemically inert.

Sealing Joints in Large Concrete Pipe *

Neoprene sleeves that overlap the joints of concrete pipe reportedly provided a simple answer to a serious leakage problem. A 36-in. concrete header pipe had developed numerous leaks at

* Condensed from *Neoprene Notebook*, June, 1955. E. I. du Pont de Nemours and Co.

the tongue and groove joints originally sealed with a troweling compound. Failure had occurred when the caulking fell out of the grooves in many places. Vibration from a nearby railroad spur probably hastened the joint compound failures.

The leaks first were repaired with different asphaltic caulking compounds and surface primers. Leaking began almost immediately due to poor adhesion to wet concrete. Finally, a gasket 12-in. wide and ⅛-in. thick was cut from a neoprene sheet. Steel bands expanded by tightening jack bolts were used to hold the gasket firmly in place. It is reported that the joints have been in service for more than a year with no difficulty.

Safetygraphs

Three new safetygraphs provide handy visual-aids for training small groups in the latest on first aid.

How to Control Bleeding illustrates the latest techniques in the field.

First Aid Treatment for Burns discusses what to do for burns as well as chemical burns of the skin and eye.

Transportation of Injured Persons covers the examination of injured persons and the use of common industrial equipment for transportation of the injured.

The safetygraphs are spiral-bound, 18 in. by 24 in. and open to form an easel. Further information and prices may be obtained from the National Safety Council, 425 N. Michigan Ave., Chicago 11, Ill.

Aluminum-Coated Steel

Aluminum alone is not strong enough, but a new aluminized steel works beautifully for use on metal roofs and other locations where the atmosphere is sufficiently corrosive to cause standard galvanized coating to fail.

The new hot-dip aluminum-coated sheet steel has been introduced by Armco after 15 yr. of testing. Known as Aluminized Steel, it combines the surface advantages of aluminum with the strength of steel. It is reported to be at least three times as resistant to corrosion as standard coated galvanized steel sheets.

Western Pennsylvania Operators Section Organized

The impetus of the clean streams program in Western Pennsylvania has led to the formation of a western section of sewage and industrial wastes operators sponsored by the Pennsylvania Sewage and Industrial Wastes Association.

An organizational meeting attended by 53 operators was held on May 24, 1955 at the Allegheny County Long Road Sewage Treatment Plant. Five meetings are scheduled per year and an invitation to join has been extended to all operators in the western part of the state.

Officers elected are:

President: Frank Varner, Altoona.
1st Vice-President: J. N. Hagerich, Grove City.
2nd Vice-President: Jim Volk, Penn. Township.
Secretary: John Yenchko, Greensburg.
Treasurer: King Hendrickson, Beaver Falls.

Orators

As the sewage treatment plant operator rises to positions of more and more responsibility he is frequently called upon to speak to both technical and lay groups. To assist the operator in this sometimes unfamiliar role, a special session in public speaking was offered to advanced operators at the 23rd Annual Short Course on Water and Sewerage held at the University of Florida, May 30 to June 4, 1955.

Graduate students in the Speech Department of the University served as instructors to small groups of operators. The sessions were well received and elicited much favorable comment from the participants.

Reviews and Abstracts*

Investigations on the Dynamics of Aeration—Effects of Some Surface Contaminants. By A. Holroyd and H. B. Parker. *Jour. Inst. Sew. Purif.* (Brit.), 280 (1952).

A laboratory study of the effects of five selected surface-active agents (Turkey Red oil, Commercial Teepol, Commercial Lissapol-N, sodium oleate, and beutonite) on the dynamics of aeration led to the following conclusions:

1. The rate of transfer of oxygen through an aeration surface is greatly influenced by the fluid flow conditions in the water adjacent to that surface. Estimations of the transfer rate, *f*, have been made for diffused air bubble surface, heaving water surface, and certain stagnant film surfaces including still water surface.

2. With addition of certain contaminants, the changes in *f* vary considerably with the character of the aeration surface.

3. Teepol, Lissapol and Turkey Red oil had the most important observable effects on aeration. Sodium oleate proved too reactive and unstable under the conditions of the experiments for reliable measurements, but indications are that soaps also would behave towards aeration like the more stable surface-active agents.

The Treatment of Gas Works Liquor in Admixture with Sewage in Percolating Filters. By W. H. Blackburn, T. G. Tomlinson and T. H. Summers. *Jour. Inst. Sew. Purif.* (Brit.), 368 (1952).

Pilot and plant-scale investigations of various gas liquor constituents on the biological oxidation of settled sewage showed that addition of the liquor in 0.5 per cent concentration by volume caused significant deterioration in the effluent quality. The greatest deterioration on small filter effluent quality was caused by addition of two crude liquor fractions, one containing thiocyanate, ammonia, and certain organic substances, the other containing carboxylic and "humic" acids. Fractions containing polyhydric phenols were more deleterious (by O.A. test) than those containing monohydric phenols, but the effect of all fractions on the effluent B.O.D. was similar.

H. P. O.

The Starting-Up of Bacteriological Filters at the Klipspruit Sewage Disposal Works. By L. E. Tucker. *Jour. Inst. Sew Purif.* (Brit.), 411 (1952).

The settled sewage as applied to four 100-ft. diameter filters containing 2½-in. media contains a high percentage of industrial wastes and is septic. The 4-hr. O.A. averages 105 p.p.m. Loading of two filters originally at 200 g.p.d. per cu.yd. was reduced after 4 weeks to 100 g.p.d. per cu.yd. because of effluent deterioration. An additional 13 weeks of dosing at the reduced rate produced a well-nitrated effluent about 70 per cent oxidized by the O.A. test. The other two new filters were loaded at 50 g.p.d. per cu.yd., which was increased to 80 after 12 weeks. After the 16th week these filters were producing an effluent of approximately the same quality as that of the other filters. H. P. O.

Synthetic Detergents and Their Effects on Sewage Treatment. By James McGill. *Jour. Inst. San. Eng.*, 53, 123 (1954).

This comprehensive literature review briefly describes and classifies synthetic detergents, outlines the theoretical difficulties and dangers expected, the difficulties anticipated as a result of laboratory or large-scale experiments, and the effects so far experienced in Southwest England. In general the observations were similar to those already described many times in the literature. H. P. O.

* Please send to Federation headquarters all periodicals, bulletins, special reports, etc., which might be suitable for abstracting in This Journal. Publications of public health departments, stream pollution control agencies, research organizations, and educational institutions are particularly desired. Address such material: Federation of Sewage and Industrial Wastes Assns., 4435 Wisconsin Ave., N.W., Washington 16, D. C.

Interrelation of Stream Regulation and Stream Pollution. By Charles M. Davidson and Milo A. Churchill. *Proc. ASCE*, 80, Sep. No. 546 (Nov., 1954).

Storage reservoirs on streams have a profound effect on their ability to assimilate pollution. Besides the sedimentation effects, there are also the effects of flow regulation, bacterial number reduction, septic sludge deposits, algae growth, smoothing out of pollutant concentrations, B.O.D. reduction, cooling, etc. The depleted oxygen state of the cooler low-level waters discharged from power dams may have a detrimental effect in stretches immediately downstream if further pollution sources exist, because the water has not had a chance to absorb sufficient oxygen to overcome the added load. H. P. O.

An Automatic Sampler for Intermittent Flows of Water. By R. Wilkinson. *Instrument Practice*, 8, 414 (1954).

A device to automatically sample stormwater flows utilizes a 5-min. sampling cycle during the first hour, followed by 1-hr. sampling until the flow falls below a predetermined level. A total of 36 samples is provided for in the mechanism. H. P. O.

BOOKS, REPORTS, AND PAMPHLETS

Report of Water Pollution Study, Mahoning River Basin. San. Eng. Div., Ohio Department of Health, Columbus, Ohio (Oct., 1954).

Progress Report No. 37. Washington Poll. Control Comm., Olympia, Wash. 26 pp. (1955).

This report covers the period June 1 to November 30, 1954.

Water Pollution Control Report for 1954. Water Poll. Control Div., Georgia Dept. of Health, Atlanta, Ga. 16 pp. (1955).

Water in Kansas. Kansas Water Resources Fact-Finding and Research Comm., Lawrence, Kans. 216 pp. (1955).

A report to the Kansas State Legislature on a state-wide survey of water resources, including recommendations for further study, conservation and development.

Tentative Standard Specifications for Cement-Mortar Liming of Water Pipelines in Place—Sizes 16 Inches and Over. AWWA c602-54T. Amer. Water Works Assn. Price, $0.25 (1954).

1954 Annual Report. Interstate Sanitation Commission, New York, N. Y. (1955).

Report on the Investigation of Leaching of a Sanitary Landfill. Publ. No. 10. State Water Pollution Control Board, Sacramento, Calif. (1954).

36th Annual Report of the Commissioner of Public Works of the City of Fitchburg, Mass.—1953. By J. M. Peirce (1954).

Cumulative Index to the First 15 Semiannual Reports to Congress 1947–53. U. S. Atomic Energy Comm. Supt. Doc., U. S. Govt. Printing Office, Washington, D. C. Price, $0.35. 118 pp. (1954).

17th Semiannual Report of the Atomic Energy Commission. U. S. Govt. Printing Office, Washington, D. C. 138 pp. (1955).

American Standards. Amer. Standards Assn., 70 E. 45th St., New York 17, N. Y. (1955).

Advanced Base Water Supply and Sanitation. U. S. Naval Civil Eng. Research and Evaluation Lab., Port Hueneme, Calif. (1953).

Sewerage Guide. Florida State Board of Health, Jacksonville, Fla. 76 pp., mimeo (1954).

Instructions for preparation, submission, and approval of projects for sewerage and sewage treatment in Florida.

Gas Piping and Gas Appliances in Buildings. NBFU No. 54, National Board of Fire Underwriters, 85 John St., New York 38, N. Y. (Sept. 1954).

Standards of NBFU for installation of gas piping and gas appliances in buildings, as recommended by the National Fire Protection Association.

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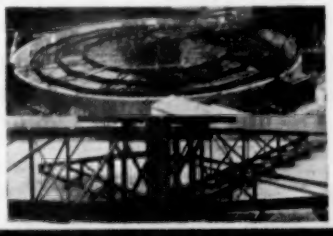

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Proceedings of Member Associations

VIRGINIA INDUSTRIAL WASTES AND SEWAGE WORKS ASSOCIATION

The 9th Annual Meeting of the Virginia Industrial Wastes and Sewage Works Association was held May 5–6, 1955 at the George Mason Hotel, Alexandria, Va. Registration reached a record total of 160, including ladies.

Excellent technical papers were present as follows:

"Water Pollution Control—One Phase of Conservation," by David B. Lee, President, FSIWA.

"Formation of a Sewer Authority," by John Pershing, Mitchell, Pershing, Shetterly and Mitchell, New York, N. Y.

"Fairfax County's Sewerage Program," by Harry Hale, Sanitary Engineer, Fairfax County Division of Sanitation, Fairfax, Va.

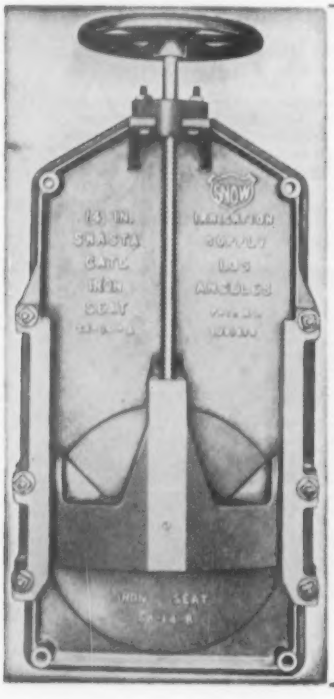
"Progress Report on Pollution Abatement in Virginia," by A. H. Paessler, Executive Secretary, Virginia State Water Control Board, Richmond, Va.

"The Counteraction of Sewage Odors," by Nicholas Post, Airkem, Inc., New York, N. Y.

"West Virginia Pulp and Paper Company's Waste Treatment Plant," by James M. Kniskern, Assistant Technical Director, West Virginia Pulp and Paper Co., Covington, Va.

"Gate Valve Design and Application," by G. J. Manahan, General Sales Manager, A. P. Smith Manufacturing Co., East Orange, N. J.

(*Continued on page 438a*)



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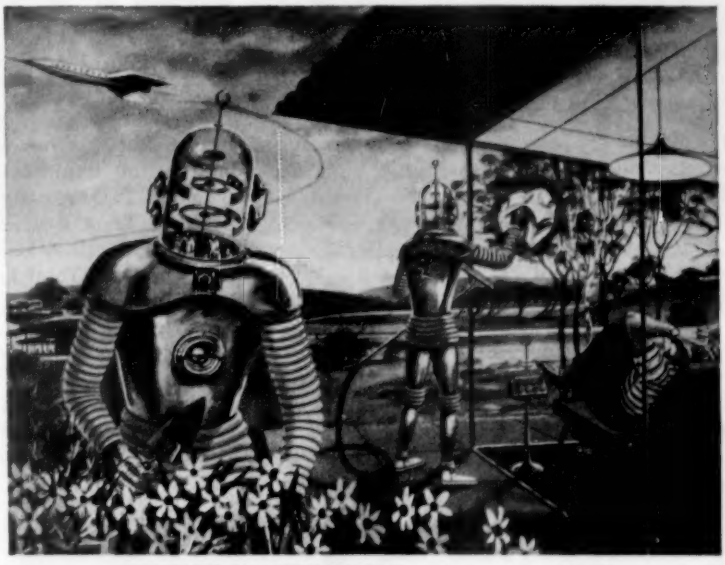

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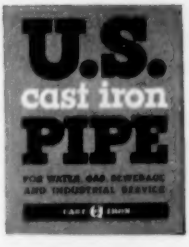

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A panel discussion on operation and maintenance of sewage treatment plants was led by E. C. Meredith, Director of Sanitary Engineering, Virginia State Department of Health, Richmond, Va.

At the Business Meeting the following officers were elected for 1955–56:

President: William R. McCann, Hopewell, Va.
Vice-President: James J. Corbalis, Jr., Alexandria, Va.
Secretary-Treasurer: J. L. Hamrick, Jr., Richmond, Va.

J. L. Hamrick, Jr.,
Secretary-Treasurer

CENTRAL STATES SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 28th Annual Meeting of the Central States Sewage and Industrial Wastes Association was held at the Kahler Hotel, Rochester, Minn., June 22–24, 1955.

Papers presented at the meeting were as follows:

"Lagooning of Industrial Wastes at the Dana Corporation, Fort Wayne, Indiana," by Charles R. Griffith, Wastes Engineer, Fort Wayne, Ind.

"Air Flotation of Sanitary Sewage," by Thomas T. Hay, Superintendent, Sewage Treatment Plant, Racine, Wis.

"The B.O.D. Test and Total Load," by A. M. Buswell, Chief, Illinois State Water Survey, Urbana, Ill.

"Recent Court Decisions Affecting Stream Pollution Control," by Warren A. Resh, Assistant Attorney General Counsel, Committee on Water Pollution, State of Wisconsin, Madison, Wis.

(*Continued on page 440a*)

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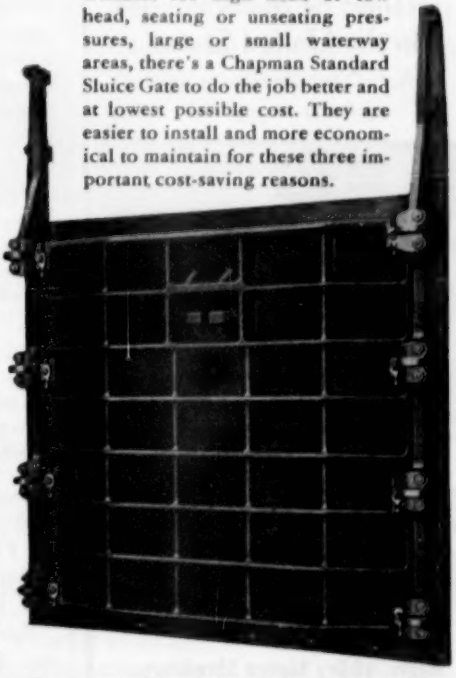

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"The Community-Wide Use of Household Garbage Grinders at Jasper, Indiana," by Frank D. Wraight, Chief, Sewage Disposal Section, Indiana State Board of Health, Indianapolis, Ind.

"The Full-Scale Modified Digestion Process Plant at Albert Lea, Minnesota," by A. J. Steffen, Sanitary Engineer, Wilson and Co., Chicago, Ill.

The afternoon session on June 23 consisted of the annual business luncheon, followed by inspection trips to the Mayo Clinic Diagnostic Building and the Rochester Sewage Treatment Works.

At the annual banquet, the Association's Gus Radebaugh Award for the best paper of 1954 was presented to Philip Morgan, Iowa City, Iowa. Awards for the best treatment plant operation reports were presented to Thomas Hay, Racine, Wis., Donald Henn, De Kalb, Ill. and W. E. Ross, Richmond, Ind. Membership merit awards went to Don Bloodgood, Lafayette, Ind., Henry Rothchild, Neenah, Wis., Kerwin Mick, Minneapolis, Minn. and Jess Dietz, Urbana, Ill.

The morning session on June 24 consisted of an "Operators' Breakfast" in which open discussion on plant operation problems was stimulated and guided by a panel consisting of Paul L. Brunner, C. C. Larson, George Martin and Kerwin Mick.

Officers elected for 1955–56 were:

President: A. J. Duvall, St. Paul, Minn.

1st Vice-President: W. E. Ross, Richmond, Ind.

2nd Vice-President: A. P. Troemper, Springfield, Ill.

3rd Vice-President: Robert Frazier, Oshkosh, Wis.

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(Continued through page 447a)

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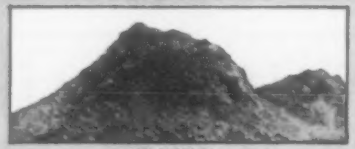

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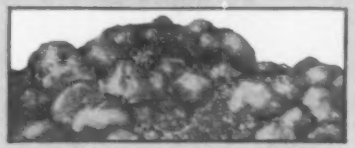

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